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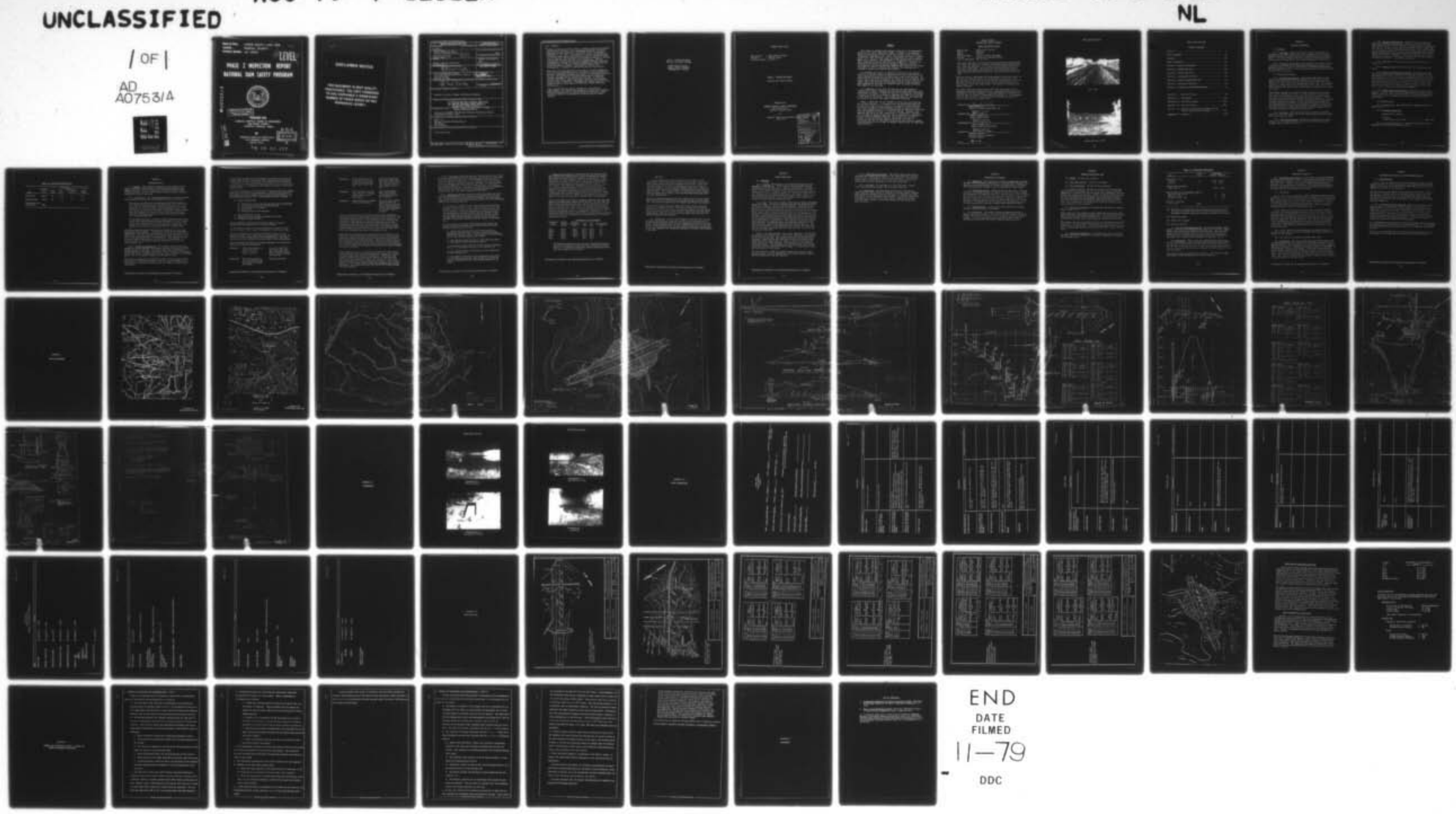

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Name Of Dam: UPPER SOUTH LAKE DAM
Location: FAIRFAX COUNTY
Inventory Number: VA. 05913

LEVEL II

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

PREPARED FOR

NORFOLK DISTRICT CORPS OF ENGINEERS
803 FRONT STREET
NORFOLK, VIRGINIA 23510

BY

DEWARD M. MARTIN & ASSOCIATES
WILLIAMSBURG, VIRGINIA
AUGUST, 1979

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20. Abstract

Pursuant to Public Law 92-367, Phase I Inspection Reports are prepared under guidance contained in the recommended guidelines for safety inspection of dams, published by the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general conditions of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

Based upon the field conditions at the time of the field inspection and all available engineering data, the Phase I report addresses the hydraulic, hydrologic, geologic, geotechnic, and structural aspects of the dam. The engineering techniques employed give a reasonably accurate assessment of the conditions of the dam. It should be realized that certain engineering aspects cannot be fully analyzed during a Phase I inspection. Assessment and remedial measures in the report include the requirements of additional indepth study when necessary.

Phase I reports include project information of the dam and appurtenances, all existing engineering data, operational procedures, hydraulic/hydrologic data of the watershed, dam stability, visual inspection report and an assessment including required remedial measures.

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

UPPER SOUTH LAKE DAM
FAIRFAX COUNTY, VIRGINIA
INVENTORY NO. VA 05913

POTOMAC RIVER BASIN

Name of Dam: : Upper South Lake Dam
Location : Fairfax County
Inventory Number: VA 05913

PHASE I INSPECTION REPORT

National Dam Safety Program

Prepared for

NORFOLK DISTRICT CORPS OF ENGINEERS
803 Front Street
Norfolk, Virginia 23510

by

Deward M. Martin & Associates, Inc.
July 1979

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of the Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established guidelines, the spillway design flood is based on the estimated "Probable Maximum Flood" for the region (flood discharges that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the design flood should not be interpreted as necessarily posing a highly inadequate condition. The design flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

BRIEF ASSESSMENT OF DAM

Name of Dam:	Upper South Lake Dam
State:	Virginia
County:	Fairfax
USGS Quad Sheet:	Vienna, Virginia, Maryland
Stream:	Tributary to Snakeden Branch
Date of Inspection:	May 30, 1979

Upper South Lake Dam is an earthfill structure 850 feet long and 55.7 feet high. The dam is in a housing development owned by Gulf Reston, Inc. in Reston, Virginia. It is used for recreational purposes and lake front development. The dam is classified as intermediate in size with a high hazard classification. The spillway is a 7-foot x 7-foot vertical shaft with a 4-foot x 4-foot box culvert through the dam.

Based on criteria established by the Department of the Army, Office of the Chief of Engineers (OCE), the Spillway Design Flood is the PMF. The spillway will pass 78% of the PMF without overtopping the dam. The PMF will overtop the dam by 0.6 feet and will cause flooding in the lower floors of the lake front houses. The spillway is therefore adjudged inadequate.

The visual inspection indicated that there was some erosion on the downstream slope which needs correction. Seepage was also noted in the embankment at the left abutment. A grouting program is currently in progress with the intent of eliminating the seepage. It is recommended that the owner, within 12 months, initiate a maintenance program to insure correction of the above items.

Prepared By: Paul Seiler
PAUL SEILER, P.E.
Deward M. Martin & Associates, Inc.

Submitted By: Original signed by
JAMES A. WALSH
JAMES A. WALSH, P.E.
Chief, Design Branch

Recommended By: Original signed by:
Carl S. Anderson, Jr.
JACK G. STARR, P.E.
Chief, Engineering Division

Approved By: Original signed by:
LTC Leonard C. Gregor
DOUGLAS L. HALLER
Colonel, Corps of Engineers
District Engineer

Date: SEP 27 1979

UPPER SOUTH LAKE DAM

Top of Dam

Downstream Face of Dam

UPPER SOUTH LAKE DAM

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SECTION 1

PROJECT INFORMATION

1.1 General:

1.1.1 Authority: Public Law 92-367, 8 Aug 72 authorized the Secretary of the Army, through the Corps of Engineers to initiate a national program of safety inspections of dams throughout the United States. The Norfolk District has been assigned the responsibility of supervising the inspection of dams in the Commonwealth of Virginia.

1.1.2 Purpose of Inspection: The purpsose is to conduct a Phase I Inspection according to the Recommended Guidelines for Safety Inspection of Dams (Appendix VI, Reference 1.) The main responsibility is to expeditiously identify those dams which may be a potential hazard to human life or property.

1.2 Project Description:

1.2.1 Dam and Appurtenances: Upper South Lake Dam is a homogeneous earth embankment 850 feet long and 55.7 feet high from the crest of the dam at elevation 350.77 to the toe of the dam at elevation 295.0. The slopes of the embankment are 3(H):1(V). The upstream slope has a 10-foot wide bench at elevation 341.5. The crest of the dam is 70 feet wide and a 4-lane curbed roadway is planned to run along its length.

The spillway consists of a 7-foot x 7-foot concrete riser structure with a crest elevation of 340.0. The inlet structure is opened on four sides to allow water to enter and each side is equipped with a bar screen to keep debris from flowing into the structure. A 4-foot x 4-foot concrete box culvert extends from the riser structure through the dam to the outlet channel. An 8-inch diameter drain pipe runs near the downstream toe of the dam and water from it is collected in an 8-inch ductile iron pipe which outlets to a manhole 170 feet downstream from the dam. There are two 24-inch diameter sluice gates in the riser structure at elevations 330 and 312.5.

1.2.2 Location: Upper South Lake Dam is in Reston, Virginia. The dam is about 0.5 miles south of the intersection of Sunrise Valley Drive and South Lake Drive.

1.2.3 Size Classification: The dam is classified as an intermediate size structure because of height (55.7 feet) and a maximum storage capacity (1406 acre feet.)

1.2.4 Hazard Classification: The dam is in the vicinity of homes both upstream and downstream. The lower floors of the lake front homes upstream from the dam will be flooded as water approaches the crest of the dam. Because of the potential property damage and loss of life, the hazard classification is high. This classification is in accordance with Section 2.1.2 of the Recommended Guidelines for Safety Inspection of Dams, published by the Department of the Army, Office of the Chief of Engineers. The hazard classification used to categorize the dams is a function of location only and has nothing to do with its stability or probability of failure.

1.2.5 Ownership: The Upper South Lake Dam is owned by Gulf Reston, Inc.

1.2.6 Purpose of Dam: The lake is used for recreational purposes.

1.2.7 Design and Construction History: The Upper South Lake Dam was designed by Massey Engineers of Fairfax, Virginia, in 1970 and constructed in 1971. There were construction control tests taken which indicated soil classification was ML-MH based on the Unified Soil Classification System. The dam was inspected by Schnabel Engineering Associates for a Phase I report in May 1978 and in December 1978 they also completed a Phase II report. Schnabel Engineering Associates' summary of conclusions and recommendations from their Phase I and II reports are included in Appendix V.

1.2.8 Normal Operating Procedures: The lake is used for recreation. The pool level is normally maintained at elevation 340.0, however, the level can be lowered manually to either elevation 330.0 or elevation 312.5 by opening one or both of the 24-inch diameter sluice gates.

1.3 Pertinent Data:

1.3.3 Drainage Area: The dam controls a drainage area of 0.6 square miles.

1.3.2 Discharge at Dam Site:

Maximum Flood - Unknown.

Spillway
pool level at top of dam . . . . 422 c.f.s.

1.3.3 Dam and Reservoir Data: Pertinent data on the dam and reservoir are shown in the following table:

Table 1.1 DAM AND RESERVOIR DATA

Item	Elevation feet m.s.l.	Reservoir Area acres	Capacity Acre feet	Capacity Watershed inches	Length miles
Top of Dam	350.77	72	1,406	44	0.7
Spillway Crest	340.00	40	813	25	0.4
Streambed at the toe of dam	295±				

SECTION 2

ENGINEERING DATA

2.1 Design: The available information, in the office of the owner, consists of plans prepared in 1970 by Massey Engineers and Reports of Phase I & II investigations prepared by Schnabel Engineering Associates in 1978. These reports were initiated to investigate loss of water in the lake.

*2.2 Construction: The following description of the dam construction is based on the May 25, 1979 Schnabel report to Gulf Reston.

The dam was constructed as a homogeneous earthfill embankment with 3(H):1(V) slopes upstream and downstream. The crest is 70 feet wide, at elevation 340.0, and will eventually have a road across it. A clay core cutoff trench varying from 2 to 8 feet was constructed along the centerline of the dam. Riprap and bedding was provided from elevation 338 to 341.5 on the upstream face for slope protection and a 10-foot wide bench was formed for a future Macadam path.

At the downstream side, a rockfill toe was constructed below elevation 300.5 with bedding and riprap extending up along the downstream earth embankment to elevation 312.0 for slope protection against wave action after a future lower lake is impounded. The normal water elevation of Lower Lake will be elevation 300.0.

An 8-inch diameter internal drain pipe was installed to collect seepage within the embankment. The drain is located in the embankment approximately 100 feet from the downstream toe of the dam (see Plate 3, Appendix I). The intent of this system was to lower the phreatic surface at the downstream side of the embankment. This drainage line is carried through a manhole just below the toe of the embankment to a discharge point in the Snakeden Branch downstream of the site of a future (lower) dam.

*2.2.1 Geologic Investigation: The original plans for the dam, prepared by Massey Engineers of Fairfax, Virginia, and dated July 1970, include sixteen boring logs. The borings were made by Penniman and Browne Engineers of Baltimore, Maryland. Figure 1 through 6 of Appendix IV show the boring locations and the boring logs, as shown in the original plans. All of the borings terminated in decomposed rock.

The depth to decomposed rock ranged from about 3 feet to about 18 feet. The shallower depth were generally in the left (north) abutment area. apparently, no laboratory or insitu tests were made in connection with the borings.

*Information provided by Law Engineering Associates of Virginia.

In June 1976, piezometers were installed to ascertain the nature of seepage from the dam. Piezometric level readings were insufficient to determine the extent and nature of the observed seepage nor was a significant correlation between the piezometric level and lake elevation determined. A summary of this study is included in the operational record.

In October and November of 1978, Schnabel conducted additional studies both in the field and in the laboratory, to determine the source of the leakage at the left abutment and to check the possibility of excessive seepage under the dam. The additional studies included:

a. Field investigations

   1) Drilling of six test borings and soil and rock sampling
   2) Field permeability test in rock formation
   3) Installation of piezometers
   4) Dye testing
   5) Additional visual observations

b. Soils laboratory testing
c. Evaluation of the current conditions of the dam
d. Proposed remedial measures

The six additional borings were located as shown in Figure B. Piezometers were installed in all six after boring.

The following account of the field investigation results is based on the December 4, 1978 report from Schnabel to Gulf Reston, Inc.

Field investigations were conducted from late October to early November in 1978. The weather was dry throughout this period, and had been for three weeks prior to the investigations. The lake level was at about elevation 338 at the time of the investigations.

The test borings indicated the following embankment and subsurface strata to the depths investigated:

Stratum A:	From ground surface to about 5 to 29 ft. depth in Borings PZ-9, 11 and 13	Brown fine SANDY SILT and sandy CLAYEY SILT (ML), embankment FILL; firm to compact density (N= 16 to 49)
Stratum A1:	From existing grade at the toe to about 10 to 14 ft depth in Borings PZ-7 and 8.	Gray rock toe FILL

Information provided by Law Engineering Associates of Virginia.

Stratum B:	Below Stratum A or A-1 to about 25 to 35 feet depth, not encountered in borings PZ-8, 9 and 12.	Brown fine SANDY SILT and sandy CLAYEY SILT (ML) with disintegrated rock layers; firm to very compact (N= 9 to 47)
Stratum C:	Below Stratum A, A-1 or B to about 12 to 40 feet depth, about 3 to 14 feet thick.	Brown DISINTEGRATED ROCK, very compact and hard density (N= 67 to 100/1")
Stratum D:	Below Stratum C to depths of all test borings.	Brown and green highly weathered PHYLLITE ROCK, highly fractured with disintegrated rock layers; (core recovery = 21 to 100%. ROD = 0 to 58%).

Bedrock underlying the dam is phyllite rock of the Wissahickon formation, based on observations of the rock cores recovered. The predominant planar feature of the phyllite is its schistosity or foliation, which dips in the rock cores from 30° to 80° from the horizontal. Observations of outcrops downstream of the left abutment indicate that the foliation strikes northwest, or about perpendicluar to the centerline of the dam. Rock Quality Designations (RQD's) measured from the NX cores recovered ranged from 0 to 58%, indicating very poor to fair quality rock.

Most fractures or joints in the rock appear to be parallel to the foliation. In core runs where recoveries were low, the cause was apparently highly weathered layers or zones within the rock which were washed out by the drilling fluid. Several excellent cores of these weathered zones were recovered. These zones, which parallel the foliation, represent zones of higher permeability and would provide direct seepage paths through the generally poor quality foundation rock of the dam.

Field studies by Schnabel included the recording of water loss or gain during drilling, and field permeability testing (the results of these studies are included in Appendix V. Additional field studies included dye testing and the results of this study are included in the operational record. The results of these field tests indicate that average permeability of Strata C and D rock is about 8.1×10^{-3} cm/sec which is much more permeable than the dam embankment. Dye testing indicated that there was seepage through the dam foundation.

*Information provided by Law Engineering Associates of Virginia.

* 2.2.2 Additional Field Observations: Wetness and numerous little ponds downstream from the rock toe were observed during the 1978 investigation. Moistened conditions of the embankment fill immediately above the downstream riprap surface were also observed, particularly at the locations of observed transverse cracking. In addition, water was observed coming out of the rock fill toe at quite a few places along the base of the toe. Erosion at the interface of the embankment fill and both the right and the left abutments has apparently worsened since the last field inspection in May 1978.

* 2.3 Operational Record: Seepage at the left abutment and excessive wetness downstream of the dam have led the owner, Gulf Reston, Inc. to undertake several inspections and studies. The most recent, conducted by Schnabel Engineering Associates, resulted in the following conclusions and recommendations:

a. The source of leakage at the left abutment has been determined from the drilling of PZ-11, which is located within the embankment and is about 3.5 feet higher in elevation than that of the "spring". The interconnection of seepage water below the disintegrated rock between PZ-11 and the "spring" caused the drilling water and dye water in PZ-11 to flow out of the "spring" under excessive water pressure from the reservoir. The lake is therefore confirmed to be the source of this leakage.

b. The existence of seepage underneath the dam through the highly fractured foundation rock has been verified from the following investigation results:

1. Almost equal piezometric levels were recorded in piezometers located at the crest just upstream and downstream from the core trench. This indicates the ineffectiveness of the installed shallow core trench.

2. The excessive water pressure at PZ-11 which was about 1.5 feet above the existing ground surface.

3. Piezometric levels recorded in PZ-7 and PZ-8 indicated about 1 to 2 feet above the base of the rockfill toe.

4. The general wetness and softness at areas downstream from the rockfill toe.

5. The numerous locations in the downstream valley where the dye water was observed. The dye water was injected into the foundation rock at PZ-9 located upstream at the crest.

*Information provided by Law Engineering Associates of Virginia.

c. Engineering properties of both soil and rock materials have been evaluated by conducting both field and laboratory testing. The results of these tests are included in Appendix IV. Both embankment fill and overburden soils may be classified as sandy clayey silt to sandy silt or type ML according to ASTM D-2487. The materials also have an average permeability value of 1.5 x 10^{-3} cm/sec. The underlying foundation rock is phyllite rock of the Wissahickon formation. The rock is generally highly weathered and highly fractured to the depth investigated. Furthermore, the rock also possesses a prominent foliation which strikes northeast or about perpendicular to the dam axis. Field permeability of 5.1 x 10^{-3} cm/sec for this highly fractured rock mass, or at least 100 times more permeable than the embankment.

After the reservoir was filled, seepage was observed from the left abutment and immediately downstream of the dam. In June 1976, the Owner of the dam, Gulf Reston, Inc., had five borings made in the dam. Piezometers were installed in these borings and periodicus readings taken for about two years in an attempt to ascertain the source of the seepage. Plate 7 in Appendix IV shows the locations of these piezometers, which are numbered PZ-2 through PZ-6 An inspection report prepared in May 1978 by Schnabel Engineering Assoc. for Gulf Reston, Inc. contained the following summary of recorded piezometric levels:

Piezometer Holes	Ground Surface (ft)	Bottom Elevations (ft)	Piezometric Level Readings High (ft)	Low (ft)	Fluctuations (ft)
PZ-2	352.0	312.0	317.0	313.5	3.5
PZ-3b	352.0	282.0	315.5	309.2	5.9
PZ-4	352.0	292.0	311.1	307.3	3.8
PZ-5	352.0	292.0	317.1	312.0	5.7
PZ-5	352.0	323.0	323 &	323 &	---
PZ-6	352.0		dry	dry	

(It should be noted that the lake level fluctuated between El 328 and El 340 for the period while the readings were taken. However, no significant correlation between piezometric level and lake elevation was observed.)

*Information provided by Law Engineering Associates of Virginia.

Dye Test

Dye was poured into test borings PZ-9 and 11 prior to the installation of the piezometers to detect the possible existence of underseepage through the rock formation below the dam. This also served the purpose of further verifying the source of seepage at the left abutment. The dye used was Sodium Flourescein. This is a dark red powder whose color changes to yellowish green when dissolved into water. The original color is restored when the dye emerges from the ground and it forms a very thin and light-reflecting film over the ponding water.

The dye was pressure injected into Test Boring PZ-11 below the depth where the loss of drilling water through the seepage area was observed. The dye water came out of the seepage area within five minutes.

Dye water was poured into test boring PZ-9 early in the afternoon on Thursday, November 2, 1978. The first evidence of dye spotted downstream from the rock toe was on Saturday, November 4, 1978, which was about two days after is was injected. There were numerous places within about a 200-foot stretch in the streambed below the rock toe where the dye water was observed.

2.4 Evaluation: Based on the information obtained from the geotechnical investigations conducted in 1978, it is recognized that a serious seepage problem exists and that remedial action should be initiated. These investigations have not considered the effect of the seepage on the dam stability nor do they contain sufficient strength data of the dam foundation and embankment materials to evaluate the stability of the dam.

*Information provided by Law Engineering Associates of Virginia.

SECTION 3

VISUAL INSPECTION

3.1 Findings:

3.1.1 General: The results of the 30 May 1979 inspection are recorded in Appendix III. At the time of the inspection the pool elevation was at 340 feet m.s.l., which is about normal. Previous Phase I and II inspections were conducted by Schnabel Engineering Associates in 1978. A summary of the conclusions and recommendations of the reports can be found in Appendix V. The ground adjacent to the abutments was covered with grass and small shrubs. Surface erosion, observed in the area of the left abutment, was probably the result of vehicles driving along the slope.

3.1.2 Dam: There are no obvious horizontal or vertical misalignments in the dam. At the left abutment about 30 feet below the crest of the dam, seepage of about 5 gpm was observed. A piezometer tube was installed by Schnabel Engineer, in 1978, just above this spot. Water was observed overflowing this tube at 15 inches above the surface. Gullies were formed in this area which are due to surface water. The water flows along an earth road down the face of the embankment at the abutment. Wetness and numerous little ponds downstream from the rock toe were observed during the 1978 investigation. Moistened conditions of the embankment fill immediately above the downstream riprap surface were also observed, particularly at the locations of observed transverse cracking. In addition, water was observed coming out of the rock fill toe at quite a few places along the base of the toe. Erosion at the interface of the embankment fill and both the right and the left abutments has apparently been worsened since the last field inspection in May 1978.

*3.1.2.1 The dam appeared to be in good condition. The crest is now serving as an unpaved road. There is no indication of settlement of the crest of the dam. The upstream slope was covered with grass and weeds. Small gullies were observed at several locations on the downstream slope, mainly near the center of the slope and adjacent to the road at the left abutment. These appeared to be created by vehicular traffic. Some sloughing was associated with these gullies indicating possible settlement and/or seepage. In some places, sand was noted on top of the riprap.

At both abutments, runoff has started erosion where the fill meets natural ground. The area downstream of the dam was damp for 200 to 300 feet although no flow was observed.

*Information provided by Law Engineering Associates of Virginia.

3.1.3 Appurtenant Structures: The outlet culvert has rocks in the apron area between the wingwalls. The rocks reduce the capacity of the culvert and should be taken from the waterway. The toe drain outlets into a manhole below the dam and then carried by pipe to a point below the lower dam which is now being constructed.

3.1.4 Spillway: The spillway is a drop inlet type, 7-foot x 7-foot concrete vertical shaft, and is in good condition.

3.2 Evaluation: The areas where gullies were detected and other signs of erosion should be repaired to prevent further deterioration and action should be taken by the owner to eliminate the vehicular traffic on the embankment and abutments. A grouting program was in progress at the time of the inspection to eliminate the seepage described above, however, it was not complete and the seepage was still occurring.

40

SECTION 4

OPERATIONAL PROCEDURES

4.1 Procedure: The reservoir is used for recreational purposes. The pool level is normally maintained at elevation 340.0, however, the level can be lowered manually to either elevation 330.0 or elevation 312.5 by opening one or both of the 24-inch diameter sluice gates.

4.2 Maintenance of Dam: Mowing is done by the owner. There is presently a grouting contract in progress to stop the seepage through the embankment and foundation material. Surface marks of cars or motorcycles, reported to be evident, were removed at the time of inspection. The dirt road at the left abutment should be removed or barricaded. The riser structure and the 4-foot box culvert appeared to be in good condition. The outlet for the box culvert, at the downstream toe of the dam, was partially blocked by debris which should be removed.

4.3 Warning System: At the present time there is no warning system or evacuation plan in operation.

4.4 Evaluation: The present grouting program will serve to address the seepage problem. The road at the left abutment should be removed and debris at the outlet of the principal spillway should be removed. A routine maintenance and inspection program should be initiated to help detect and correct problems which may occur.

SECTION 5

HYDRAULIC/HYDROLOGIC DATA

5.1 Design: No data was available.

5.2 Hydrologic Records: None were available.

5.3 Flood Experience: No records were available.

5.4 Flood Potential: The PMF and 1/2 PMF were developed and routed through the reservoir by use of the HEC-I computer program (Reference 2, Appendix VI), and appropriate unit hydrograph, precipitation, and storage-outflow data. Clark's Tc and R coefficients for the local drainage area were estimated from basin characteristics. The rainfall applied to the developed unit hydrograph was obtained from a U S Weather Bureau Publication (Reference 3, Appendix VI). Losses were estimated at an initial loss of 1.0 inch and a constant loss thereafter of 0.05 inch/hour.

5.5 Reservoir Regulation: Pertinent dam and reservoir data are shown in Table 1.1.

Water flows over the spillway to Lower South Lake in the event water in the reservoir rises above elevation 340.0. The spillway is a drop inlet type, 7-foot x 7-foot vertical shaft. A 4-foot x 4-foot box culvert from the drop inlet runs through the dam to the Lower Lake.

The storage curve was calculated by use of U S Geological Survey Quadrangle Maps. Rating curves were developed for the spillway and non-overflow section of the dam. In routing hydrographs through the reservoir, it was assumed that the initial pool level was at the spillway crest.

5.6 Overtopping Potential: The probable rise of the reservoir and other pertinent information on reservoir performance is shown in the following table:

Table 5.1 RESERVOIR PERFORMANCE

Item	Normal flow	Hydrograph 1/2 PMF	PMF (a)
Peak flow, c.f.s.			
Inflow	3	3,810	7,619
Outflow	-	420	1,631
Maximum pool elevation feet, m.s.l.		346.2	351.4
Non-overflow section (elevation 350.77)			
Depth of flow, ft (b)		--	0.6
Duration, hours		--	3.3
Velocity, f.p.s. (d)		--	3.6
Tailwater elevation, feet, m.s.l.	300±		

(a) The PMF is an estimate of flood discharge that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region.

(b) Normal flow depth

(c) Normal flow velocity

(d) Critical velocity

5.7 Reservoir Emptying Potential: Two 24-inch diameter sluice gates in the drop inlet spillway at elevations 312.5 and 330, respectively, are available for dewatering the reservoir. The gates will permit withdrawal of about 110 c.f.s. with the reservoir level at the crest of the spillway and essentially dewater the reservoir to elevation 311.5 in about 10 days assuming an inflow of 3 c.f.s.

5.8 Evaluation: Based on the size (intermediate) and hazard (high) classifications, the recommended Spillway Design Flood is PMF. The spillway will pass 78% of the PMF without overtopping the dam. The PMF will overtop the dam for 3.3 hours and reach a maximum of 0.6 feet over the top of the dam, with an average critical velocity of 3.6 feet per second.

Conclusions pertain to present day conditions. The effect of future development on the hydrology has not been considered.

SECTION 6

STRUCTURAL STABILITY

*6.1 Foundation and Abutments: Soils found in the dam foundation and abutments are generally clayey silts which are generally about 10 feet thick and overlying a layer of disintegrated rock which is about 3-14 feet thick. Bedrock consisting of highly weathered phyllite underlies the disintegrated rock stratum. Both strata (disintegrated rock and bedrock) are highly permeable. The cutoff trench generally extended into the disintegrated rock stratum.

6.2 Embankment:

*6.2.1 The dam is a homogeneous earth embankment constructed with ML-MH type soils. The embankment is 850 feet long, 55.7 feet high, and with a 70 foot wide crest. The slopes of the embankment are 3(H):1(V). The upstream slope has a 10-foot wide bench at elevation 341.5 and 5 feet of riprap below a path at this elevation. There is a stone toe at elevation 300.5 on the downstream side with a filter and riprap extending from the stone toe to elevation 312.0.

*6.2.2 While the crest of the dam is wider than usual and the relatively flat slopes indicate a probable stable design, no studies have been made to confirm this nor is it known if any analyses were performed in connection with the original design. The effect of the seepage at the left abutment and in the dam foundation and the potential for piping has not been evaluated. The cause of the displaced filter bedding under the downstream riprap has not been determined. It is also unknown whether or not any stability analyses have been performed for this dam that consider the effect of the water level of the lower lake.

*6.3 At the time of the dam inspection, a remedial program to install a grout curtain at the left abutment and central portions of the dam was in progress.

*6.4 The dam site is located in Seismic Risk Zone 1.

*6.5 Evaluation: The visual inspections and interim Phase I and II investigations have been adequate to determine the nature of the observed seepage. The performance of the grouting program in progress should be monitored by a qualified registered engineer to determine if any additional studies are necessary. The cause of the displaced filter bedding material under the downstream riprap should be determined. Stability analyses should also be conducted on the downstream slope that consider the influence of high water levels from the lower lake. These analyses should include, but not be limited to, an analyses of rapid drawdown of the lower lake.

*Information provided by Law Engineering Associates of Virginia.

SECTION 7

ASSESSMENT AND REMEDIAL MEASURES/RECOMMENDATIONS

7.1 Dam Assessment:

*7.1.1 The dam appears to be in sound condition except for minor erosion problems on the downstream face, some improper bedding of the riprap, the seepage at the left abutment and under the dam, and rock debris at the outlet of the spillway.

Based on criteria established by the Department of the Army, Office of the Chief of Engineers (OCE), the Spillway Design Flood is the PMF. The spillway will pass 78% of the PMF without overtopping the dam. The PMF will overtop the dam by 0.6 feet and will cause flooding in the lower floors of the lake front houses. The spillway is therefore adjudged inadequate.

*7.2 Recommended Remedial Measures: After the grouting program, now in progress, is completed, the performance of the grouting program should be studied by a registered engineer to determine if any additional studies are necessary. Stability analyses should be performed which include the effects of future high water levels on the downstream slope. The minor erosion and gullies on the downstream slope at the abutments should be repaired. Should sloughing and gully formation persist on the downstream slope, a professional engineer should be retained to further study the problems.

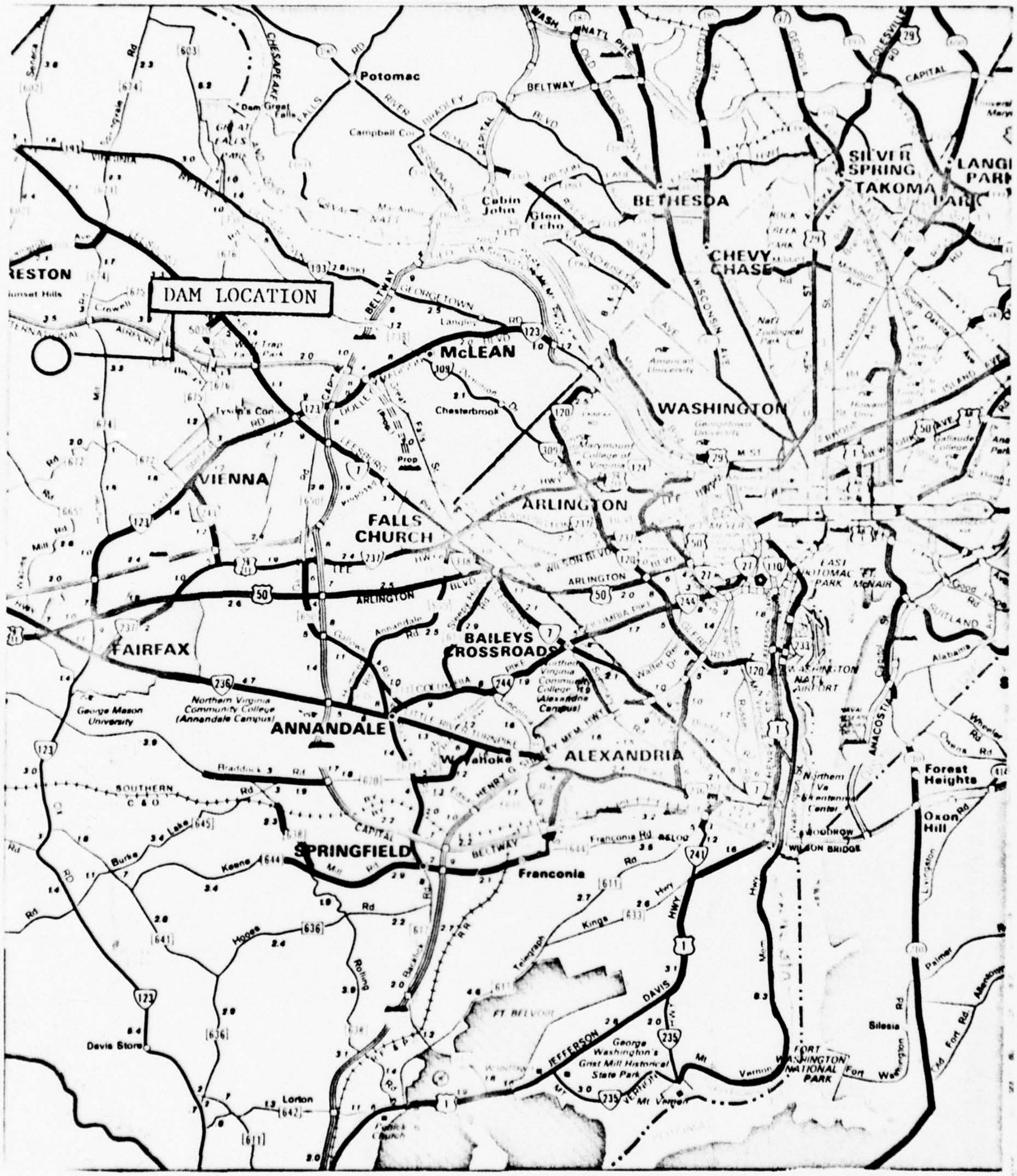
It is recommended that the toe drain riprap be removed in several places to determine if the filter bedding has been washed out. If it is evident that the bedding material has been washed out, it should be replaced as necessary.

Since it is a requirement that stability analyses be available, it is recommended that such analyses be performed by a registered engineer.

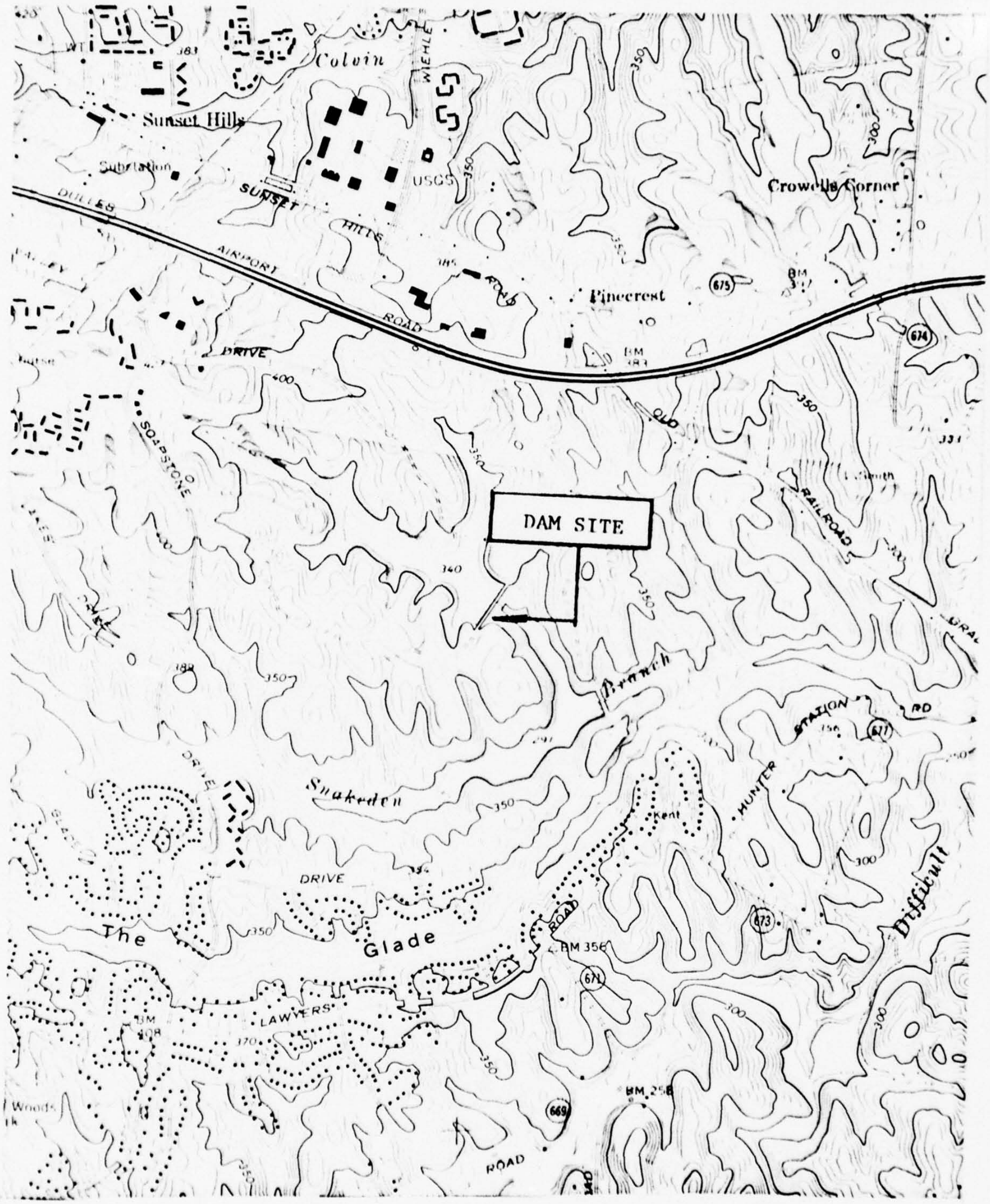
*Information provided by Law Engineering Associates of Virginia.

APPENDIX I

MAPS AND DRAWINGS



REGIONAL MAP
UPPER SOUTH LAKE DAM



VIENNA, VA.—MD.

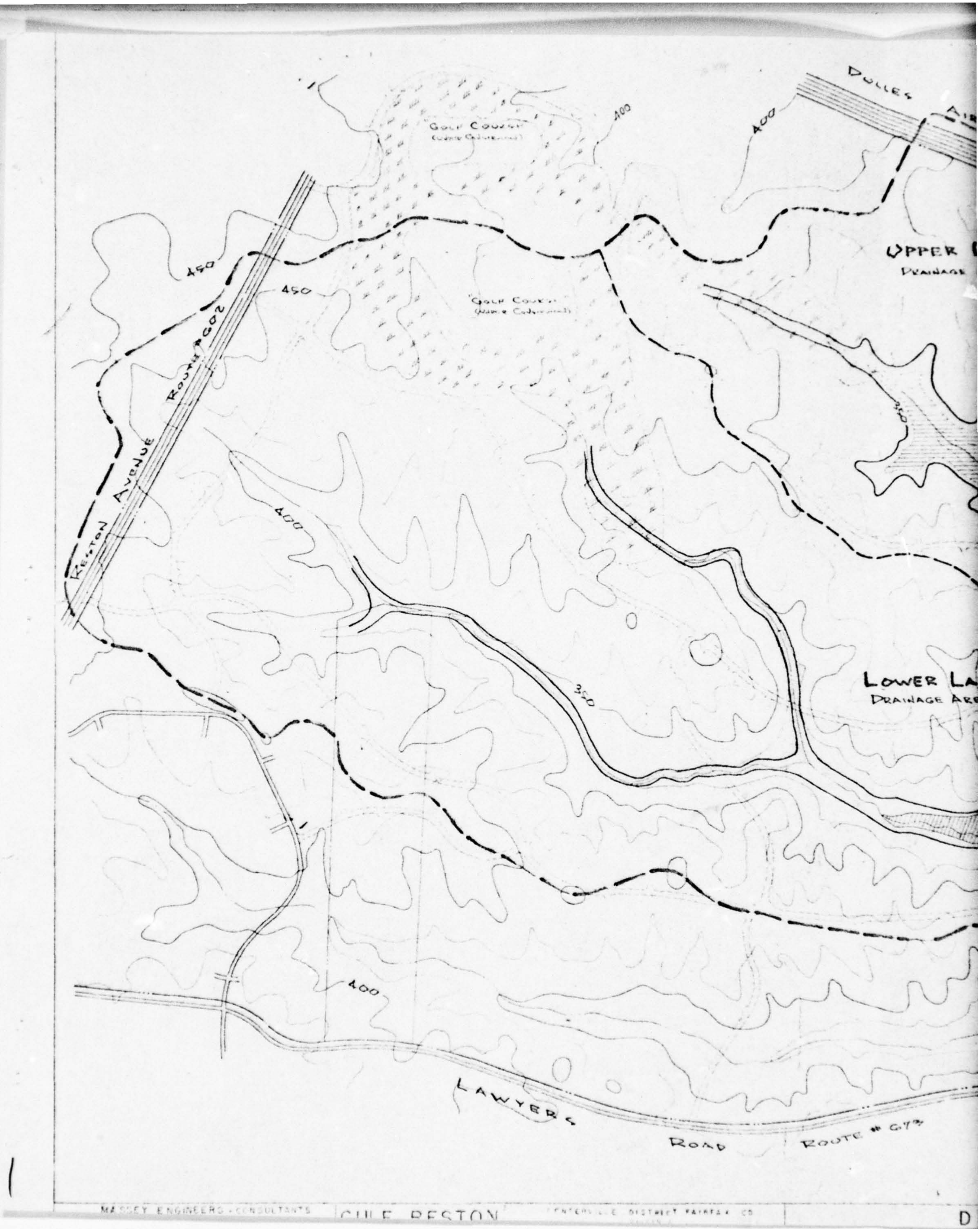
N3852.5—W7715/7.5

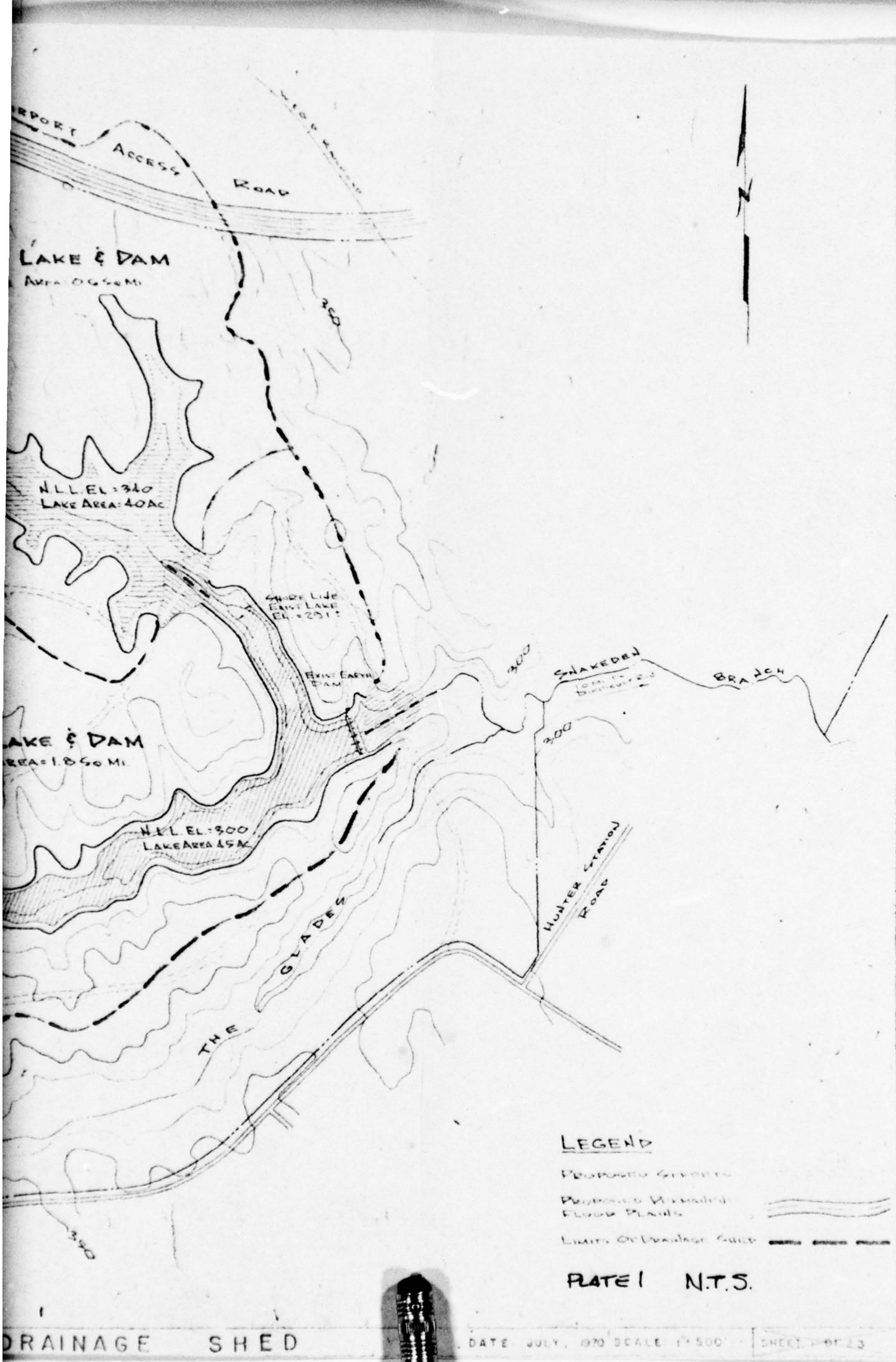
1973

AMS 5561 IV NE—SERIES V834

Scale: 1" = 2000'
10' Contours

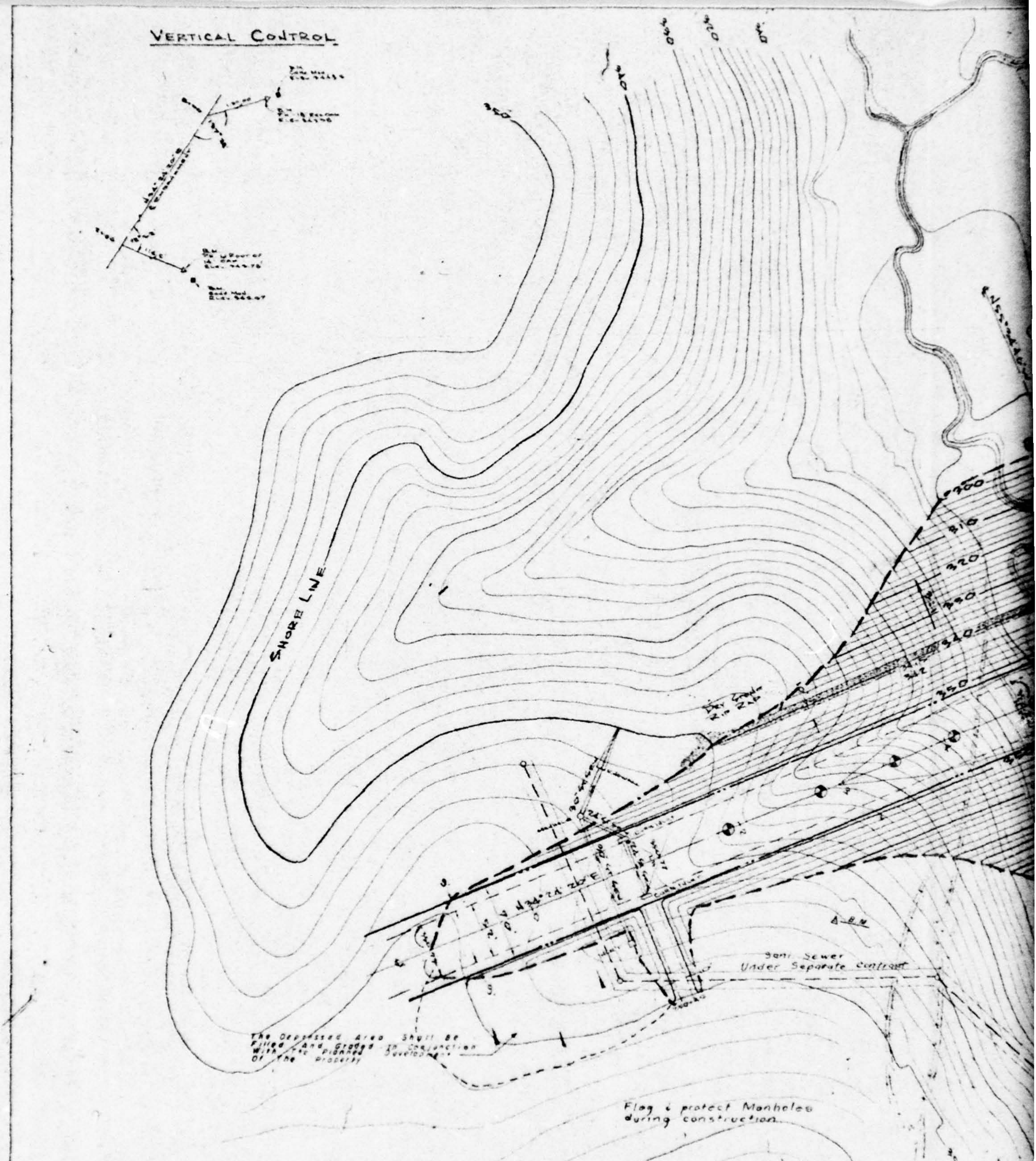
VICINITY MAP
UPPER SOUTH LAKE DAM


DULLES
UPPER
DRAINAGE
GOLF COURSE
GOLF COURSE
400
400
450
450
350
350
400
RESTON AVENUE ROUTE #602
LOWER
DRAINAGE
400
LAWYERS ROAD ROUTE # 673
MASSEY ENGINEERS - CONSULTANTS
GULF RESTON
CENTERVILLE DISTRICT FAIRFAX CO



DRAINAGE SHED DATE JULY, 1970 SCALE 1"=500' SHEET OF 23

2


VERTICAL CONTROL
SHORE LINE
Sani. Sewer
Under Separate Contract
The Depressed Area Shall Be Filled And Graded In Conjunction With The Planned Development Of The Property
Flag & protect Manholes during construction.
HORIZONTAL CONTROL
℄ STATION 0+00 N463,567.000 E2,332,062.395
℄ STATION 5+50 N464,068.283 E2,332,542.793

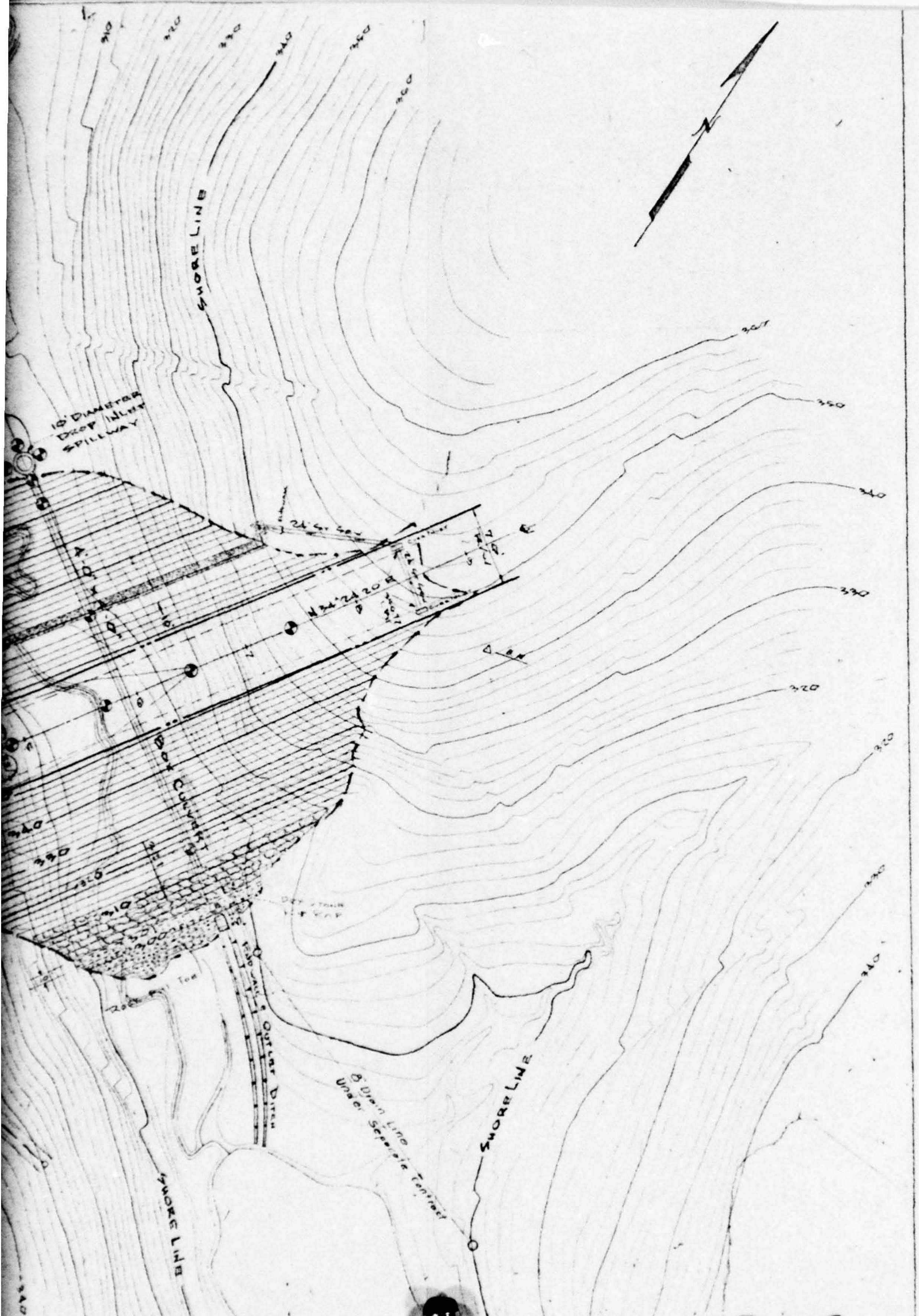
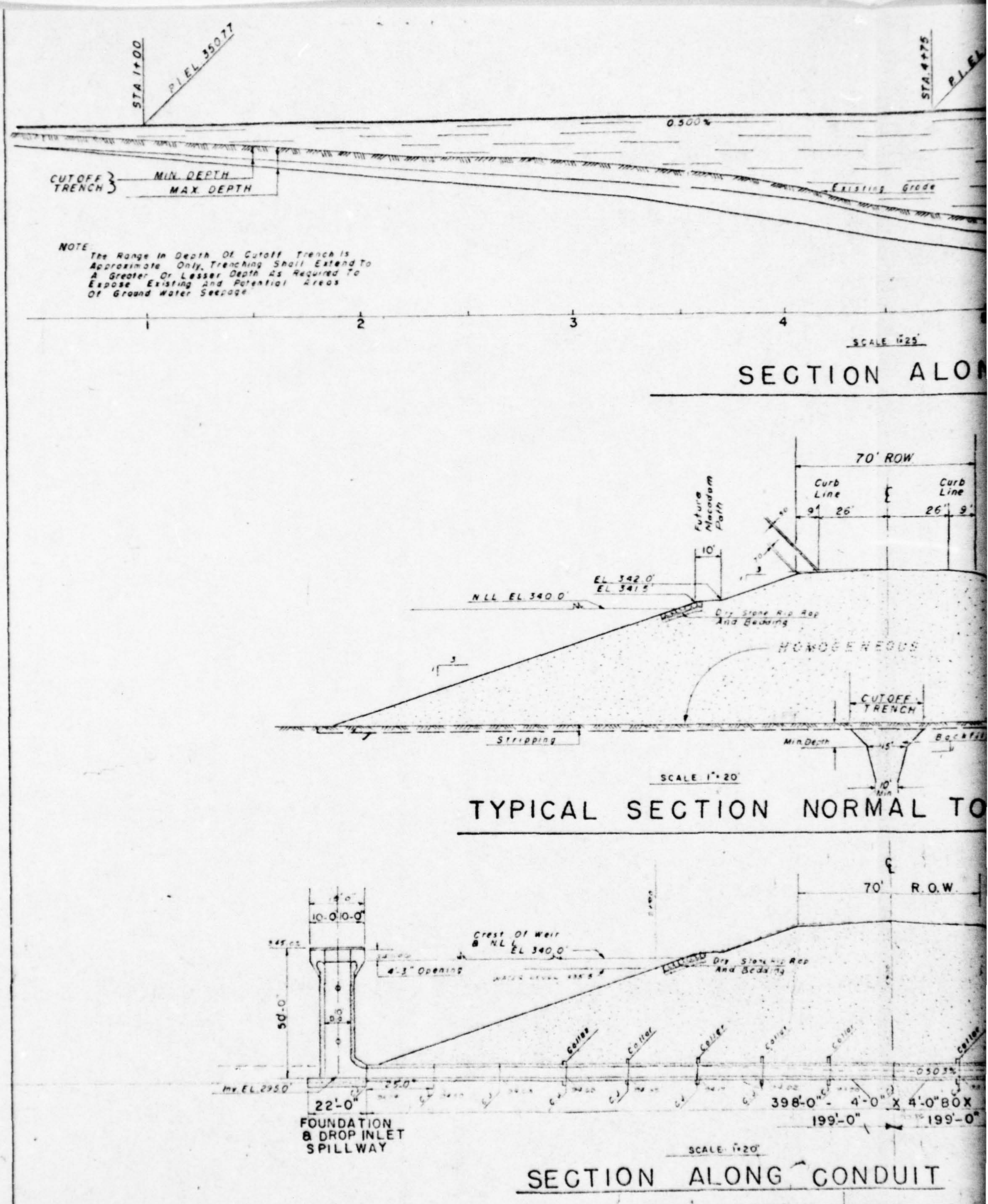
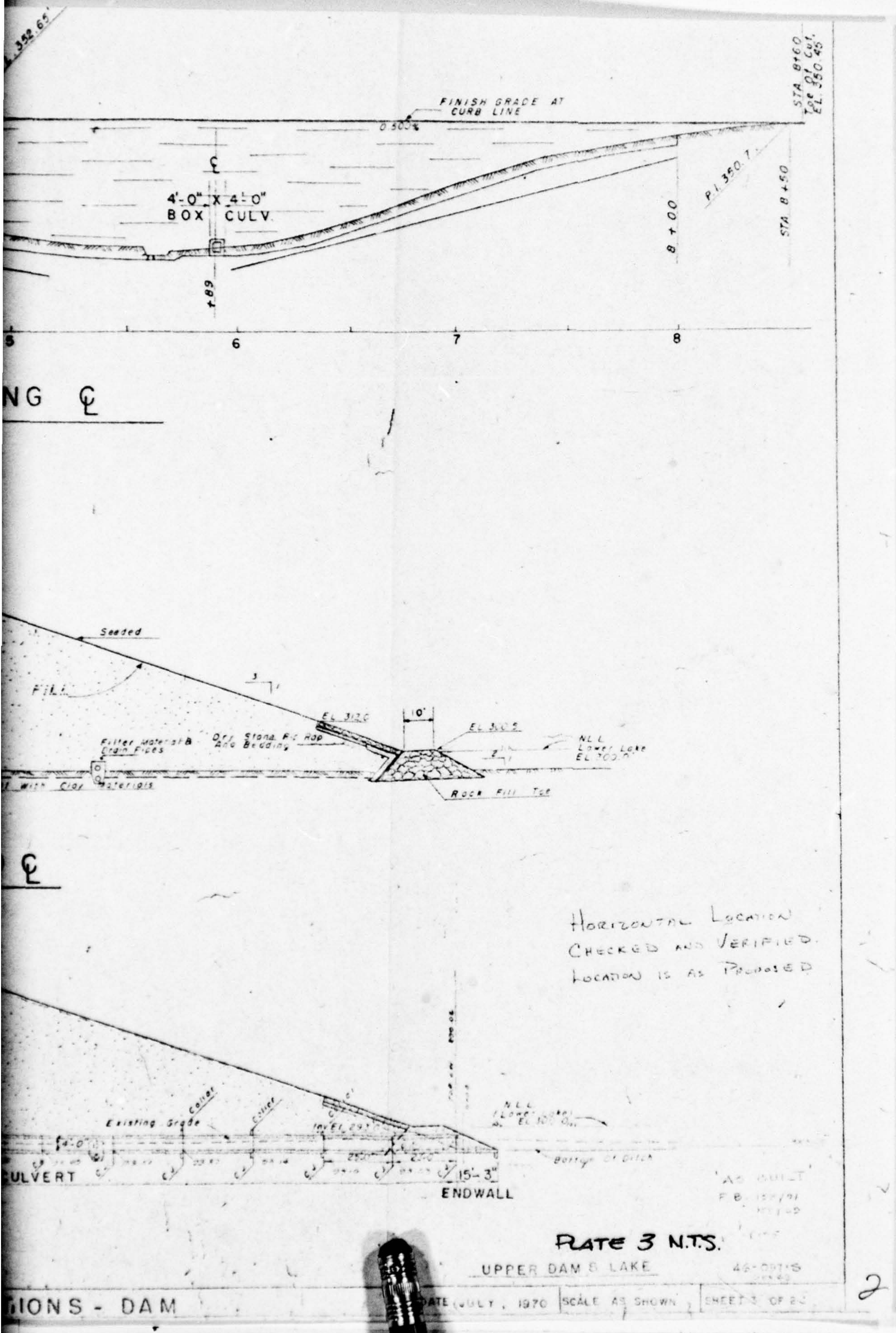


PLATE 2
N.T.S.
UPPER DAM & LAKE



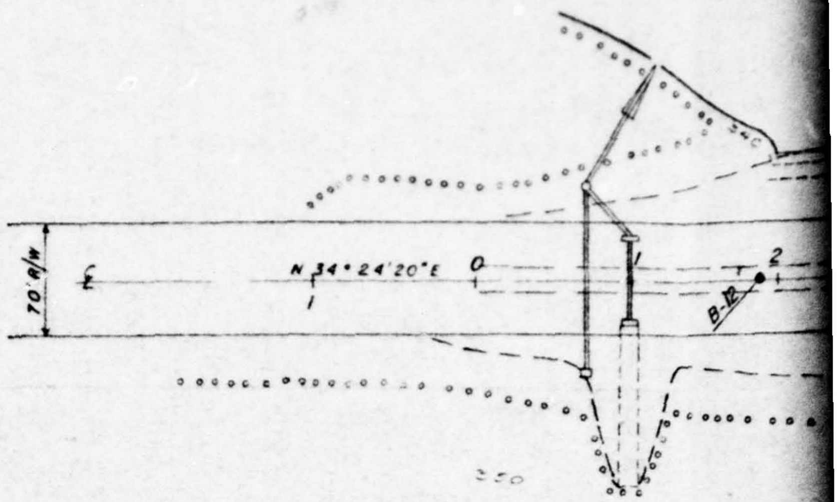
MASSEY ENGINEERS CONSULTANTS | GULF-RESTON | CENTERVILLE DISTRICT FAIRFAX CO. VIRGINIA | SEC

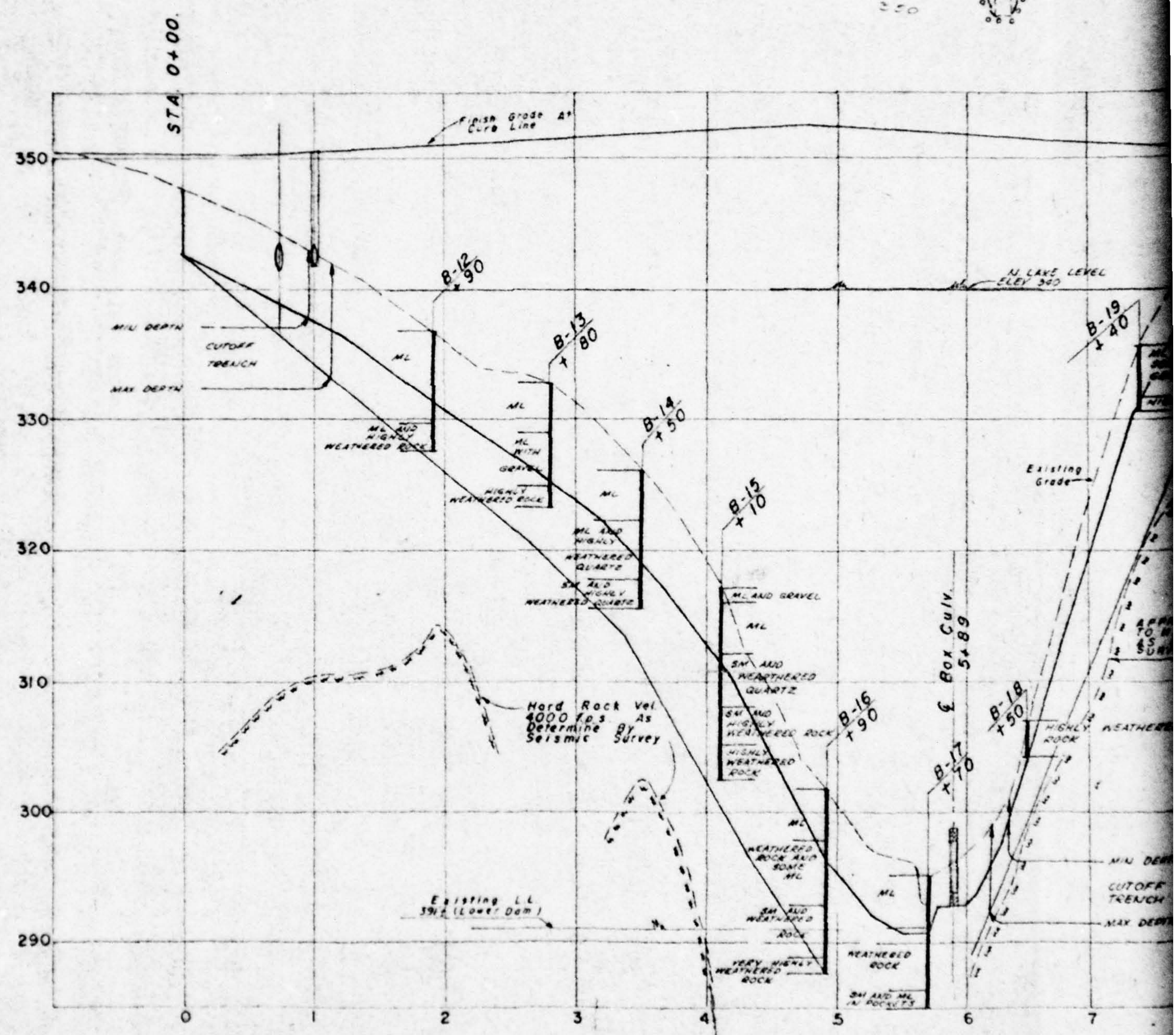

FINISH GRADE AT CURB LINE
0.300%
4'-0" X 4'-0" BOX CULV.
+89
8 + 00
P.I. 350.7
STA 8+50
STA 8+60
Toe of Cut
EL. 350.45
6
7
8
NG ℄
Seeded
FILL
EL 312.0
10'
EL 300.5
Filter Material & Drain Pipes
Dry Stone Rip Rap And Bedding
NLL
Lower Lake
EL 300.0
Rock Fill Toe
℄
Horizontal Location
Checked and Verified.
Location is as Proposed
Existing Grade
Collar
NLL
(Lower Lake)
EL 300.0
Bottom of Ditch
15'-3"
CULVERT
ENDWALL
'AS BUILT'
PLATE 3 N.T.S.
UPPER DAM & LAKE
IONS - DAM
DATE JULY, 1970
SCALE AS SHOWN
2

SOIL CLASSIFICATION

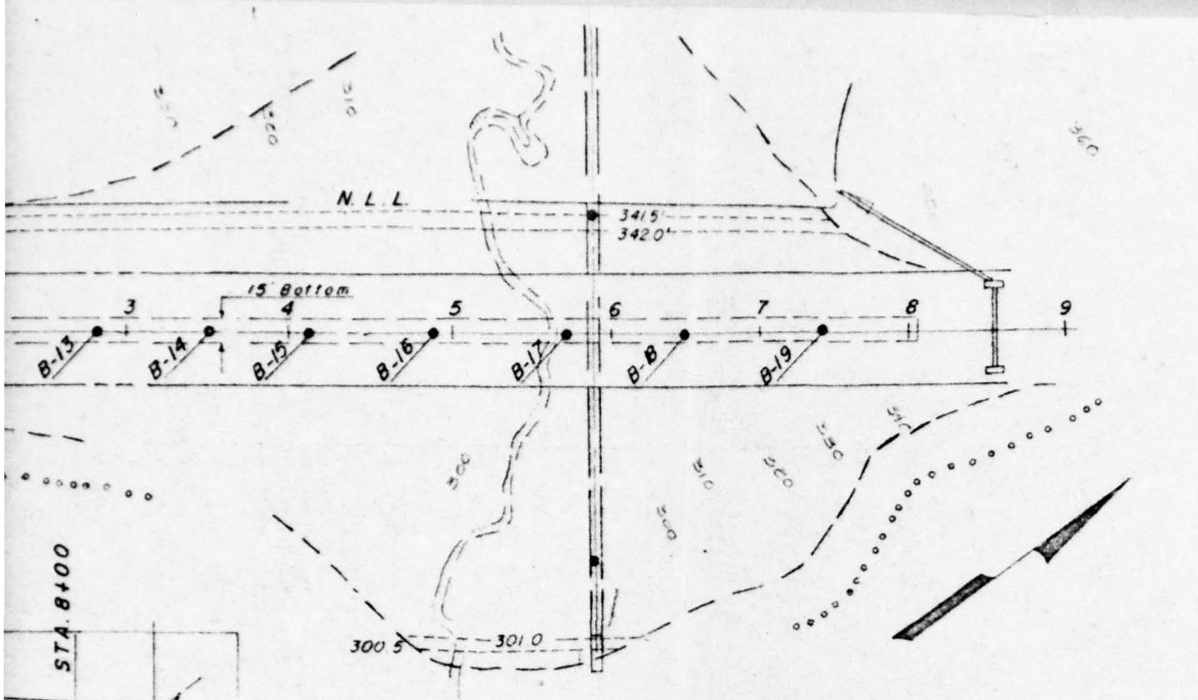
ML	Clayey silt with slight plasticity
SM	Silty sand
SP	Poorly graded sand with few fines
GP	Poorly graded gravel or gravel sand
SM-SC	Silty clayey sand mixture
SC	Clayey sand

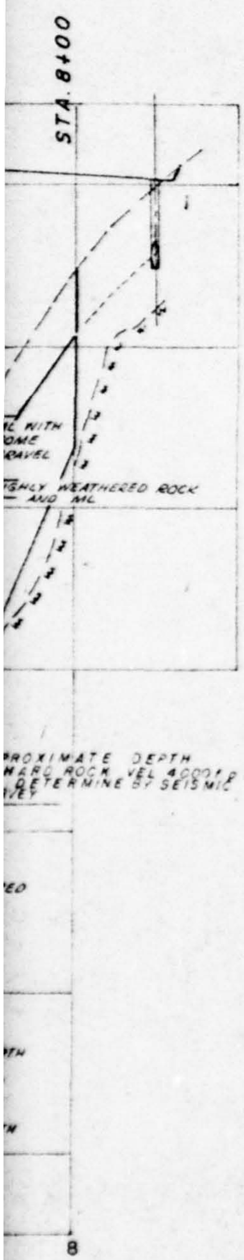
NOTE: The soils of this site are predominantly ML; the soils are derived from micaceous schist or highly weathered rock.





CENTERVILLE DISTRICT FAIRFAX CO.





SOIL BORING LOG

B-12 GRND ELEV ±336.94

DEPTH FT	CASE B/1'	SPOON B/6"	FIELD DESCRIPTION
0			0'
1	3	3 4 7	brown clayey silt
2	3		
3	9		
4	12		4'
5	14	10 14 21	brown clayey silt
6	35		
7	42		7'
8	81		brown decomposed rock
8.5	160/6"		
8.8		100/3"	

B-13 GRND ELEV ±338.57

DEPTH FT	CASE B/1'	SPOON B/6"	FIELD DESCRIPTION
0			0'
1	3	3 5 7	brown clayey silt
2	3		
3	9		
4	18		4'
5	25	12 23	brown clayey silt & decomposed rock
6	32		
7	47		
8	69		8'
9	112		brown decomposed rock
9.4		100/5"	

B-14 GRND ELEV ±326.12

DEPTH FT	CASE B/1'	SPOON B/6"	FIELD DESCRIPTION
0			0'
1	3	3	reddish brown clayey silt
2	3	5	
3	9		
4	15		4'
5	25	12 20	brown clayey sand & quartz
6	35	29	
7	42		
8	66		8'
9	81		brown decomposed rock
10.5		30 37 61	

B-15 GRND ELEV ±317.56

DEPTH FT	CASE B/1'	SPOON B/6"	FIELD DESCRIPTION
0			0'
1	2	1 2 4	brown clayey silt
2	4		
3	9		
4	14		4'
5	25	7 7 9	brown clayey silt
6	27		
7	32		
8	40		
9	46		9'
10	51	12 30 41	brown clayey silt & quartz
11	56		
12	80		12'
13	112		brown decomposed rock
13.5	70/6"		
		35 56 70	

B-16 GRND ELEV ±302.08

DEPTH FT	CASE B/1'	SPOON B/6"	FIELD DESCRIPTION
0			0'
1	1	1 1 4	brown clayey silt
2	2		
3	9		
4	21		4'
5	26	25 12 10	brown clayey silt & rock fragments
6	35		
7	37		
8	41		
9	60		9'
10	26	35 20 25	brown silty sand
11	35		
12	58		
13			13'
13.5			brown decomposed rock
		70 100/5"	

B-17 GRND ELEV ±295.53

DEPTH FT	CASE B/1'	SPOON B/6"	FIELD DESCRIPTION
0			0'
1	1	1	brown silty clay
2	1		
3	1		
3.5			3.5'
4	4		green decomposed rock
5	20	25 35 47	
6	25		
7	37		7'
8	39		brown silty clay
9	56		9'
		25 66 100	brown decomposed rock

PLATE 4 N.T.S.

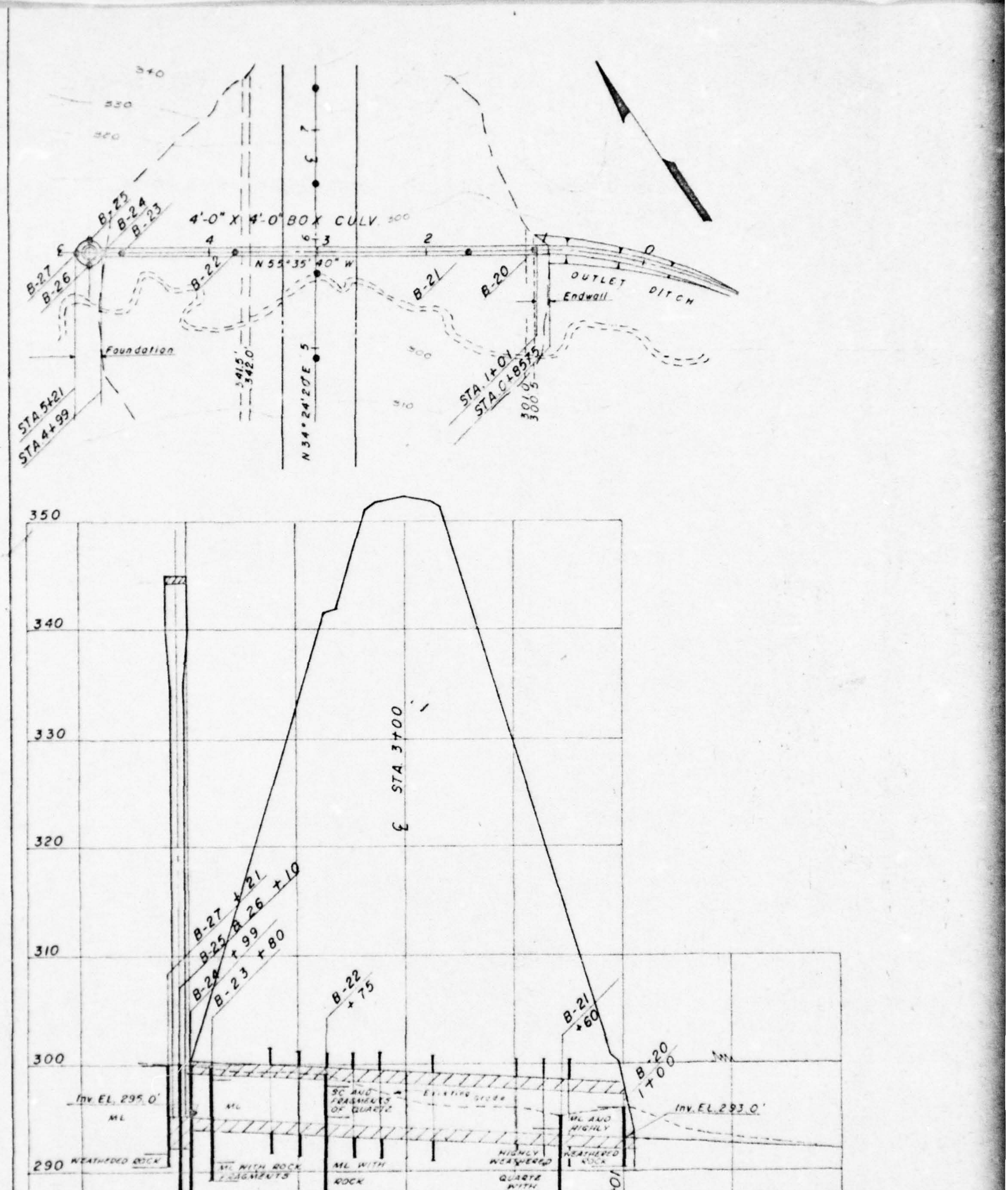
UPPER DAM 8 LAKE

46-055-S
Texas

VER [illegible]

SOIL BORINGS

2


4'-0" X 4'-0" BOX CULV.
N 55° 35' 40" W
N 34° 24' 20" E
B-27
B-26
B-25
B-24
B-23
B-22
B-21
B-20
OUTLET DITCH
Endwall
Foundation
STA 5+21
STA 4+99
STA. 1+01
STA. 0+85.75
341.5'
342.0'
301.0
300.5
350
340
330
320
310
300
290
STA 3+00
B-27 +21
B-25 B-26 +10
B-24 +99
B-23 +80
B-22 +75
B-21 +60
B-20 1+00
Inv. EL. 295.0'
Inv. EL. 293.0'
ML
WEATHERED ROCK
ML WITH ROCK FRAGMENTS
SC AND FRAGMENTS OF QUARTZ
Existing grade
ML WITH ROCK FRAGMENTS
SM WITH ROCK FRAGMENTS
HIGHLY WEATHERED QUARTZ WITH TRACE OF ML
ML AND HIGHLY WEATHERED ROCK
STA 1+01
CENTERVILLE DISTRICT FAIRFAX CO.

SOIL BORING LOG

B-18 GRND ELEV ±308.80

DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	3	3, 16, 12	brown clayey silt & decomposed rock
2	12		
3	72		3'
3.9		35, 100/3"	green decomposed rock

B-19 GRND ELEV ±334.83

DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	3	3, 4, 5	brown clayey silt
2	5		
3	12		
4	16		4'
5	43	12, 16, 36	yellowish brown clayey silt
5.5	114/6"		5.5'
5.8			brown decomposed rock
		100/3"	

B-20 GRND ELEV ±295.57

DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	5	3, 7, 12	brown silty clay & decomposed rock
2	12		
3	35		
4	61		4'
4.5		36, 100/5"	brown decomposed rock

B-21 GRND ELEV ±294.65

DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	3	3, 4, 6	gray micaceous clayey silt
2	4		
3	6		
4	9		4'
5	25	25, 30, 42	brown silty clay & quartz rock
6	42		
7	65		
8	78		
9	95		9'
		27, 42, 86	green decomposed rock

B-22 GRND ELEV ±298.78

DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	2	1, 2, 5	Topsoil
2	3		
3	18		
4	18		4'
5	21	7, 9, 10	brown silty clay & quartz
6	24		
7	19		
8	29		
9	37		9'
10	52	10, 12, 12	brown decomposed rock
11	75		
12	81		
13	100		13'
13.8		66, 100/4"	brown decomposed rock

B-23 GRND ELEV ±299.50

DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	3	3, 3, 5	brown silty sandy clay
2	4		
3	6		
4	8		4'
5	11	11, 10, 10	brown clayey silt & decomposed rock
6	11		
7	10		
8	25		8'
9	51		brown decomposed rock
		37, 51, 50	

B-24 GRND ELEV ±300.05

DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	2	2, 2, 3	brown sandy clay
2	3		
3	6		
4	9		4'
5	12	12, 11, 14	brown decomposed rock
6	14		
7	15		
8	16		
9	21		9'
10	46	35, 37, 48	brown sandy clay & decomposed rock
11	57		
12	92		
12.3	80/3"	100/3"	12.3'
			gray decomposed rock

B-25 GRND ELEV ±301.08

DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	2	2, 6, 4	brown silty clay
2	7		
3	6		
4	7		4'
5	9	6, 7, 12	brown sandy silt & decomposed rock
6	13		
7	26		
8	35		8'
9	109		brown decomposed rock
9.8		76, 100/3"	

B-26 GRND ELEV ±299.39

DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	7	1, 1, 1	brown silty clay
2	8		
3	8		
4	15		4'
5	17	8, 16, 16	brown decomposed rock & quartz
6	24		
7	14		
8	15		
9	40		9'
10	87	25, 64, 68	brown decomposed rock
11	90		11'
12	120		brown decomposed rock
12.5	97/6"		
		92, 100/2"	

B-27 GRND ELEV ±299.90

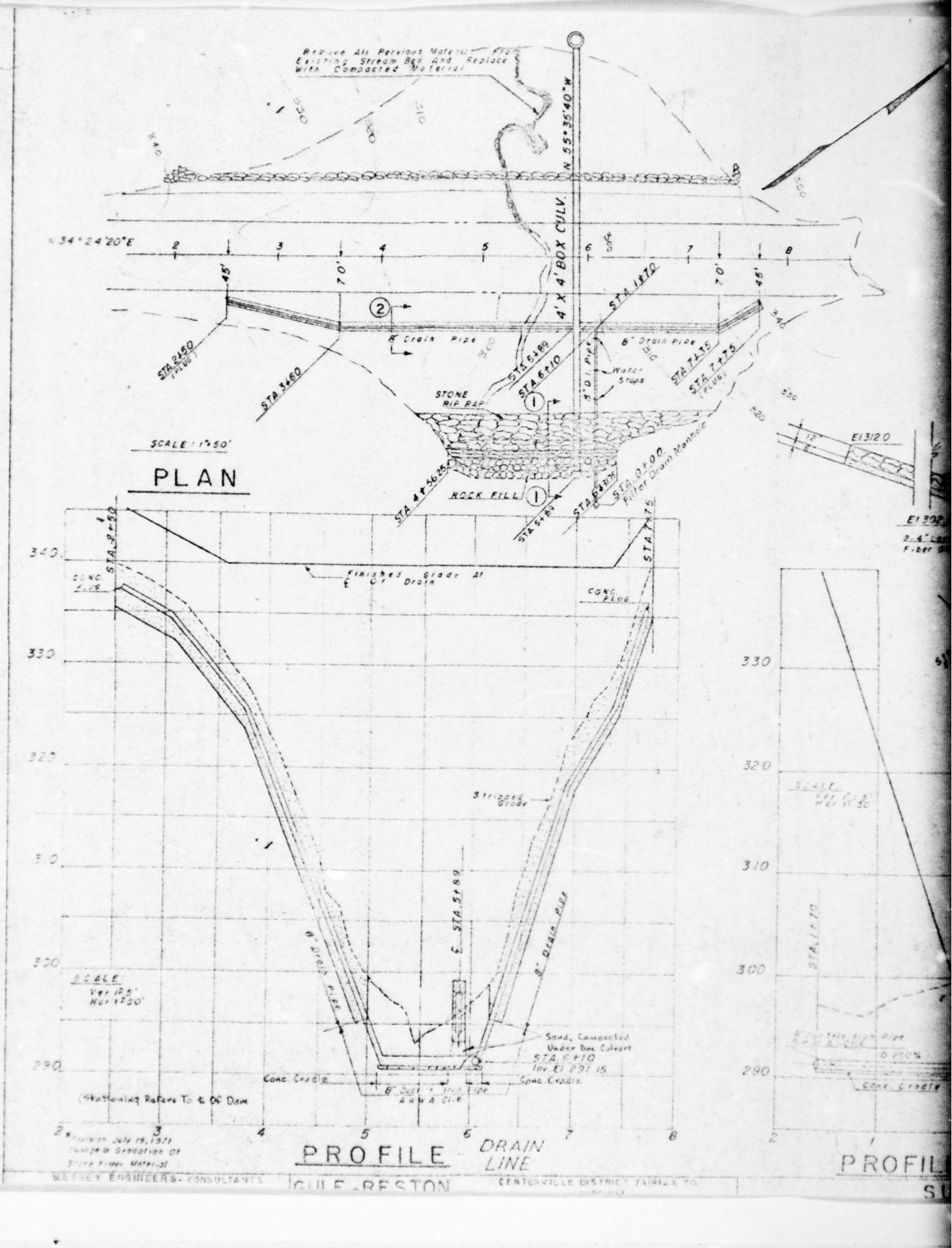
DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	3	2, 3, 5	brown silty clay
2	2		
3	3		
4	11		4'
5	14	10, 14, 22	brown sandy silt & decomposed rock
6	18		
7	15		
8	10		8'
9	41		brown decomposed rock
9.4		100/4"	

PLATE 5 N.T.S.

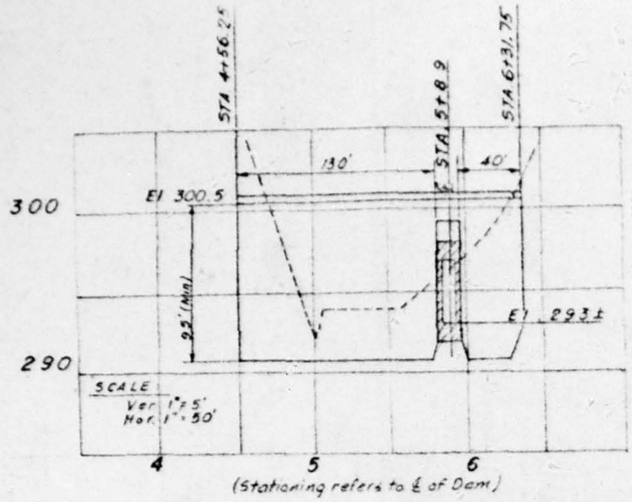
UPPER DAM B LAKE

BORINGS | DATE: JULY 197[illegible] | SCALE VER 1"=5' | SHEET [illegible]

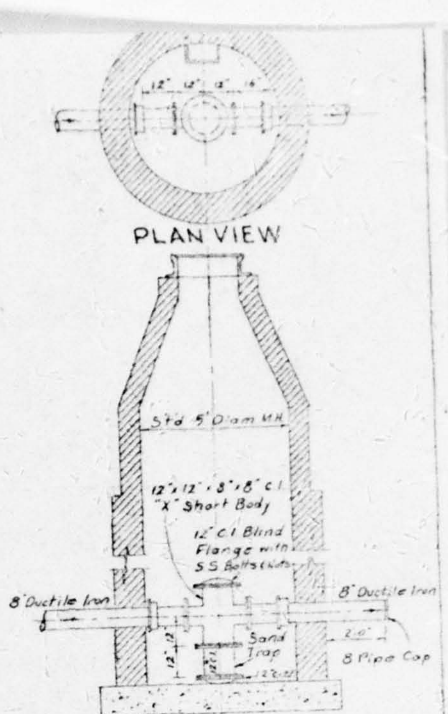
2



Remove All Pervious Material From Existing Stream Bed And Replace With Compacted Material
N 34° 24'20"E
N 55°35'40"W
4' X 4' BOX CULV.
45'
70'
70'
45'
8" Drain Pipe
8" Drain Pipe
STA 2+50 (PLUG)
STA 3+60
STA 5+89
STA 6+10
STA 1+70
8" D.I. Pipe
Water Stops
STA 7+35
STA 7+75 (PLUG)
STONE RIP RAP
SCALE: 1"=50'
PLAN
STA 4+56.25
ROCK FILL
STA 5+89
STA 6+33
STA 0+00 Filter Drain Manhole
El 312.0
12"
STA 2+20
STA 7+75
Finished Grade At ℄ Of Drain
CONC. PLUG
CONC. PLUG
Stripped Grade
STA 5+89
8" Drain Pipe
8" Drain Pipe
SCALE: Ver 1"=5' Hor 1"=50'
Sand, Compacted Under Box Culvert
STA 6+10 Inv El 291.15
Conc. Cradle
Conc. Cradle
8" Ductile Iron Pipe A.W.W.A. C1-6
(Stationing Refers To ℄ Of Dam)
Revision July 19, 1971 Change In Gradation Of Stone Filter Material
340
330
320
310
300
290
2
3
4
5
6
7
8
PROFILE - DRAIN LINE
MASSEY ENGINEERS - CONSULTANTS
GULF-RESTON
CENTERVILLE DISTRICT FAIRFAX CO.
330
320
310
300
290
STA 1+70
0.250%
Conc. Cradle
PROFIL

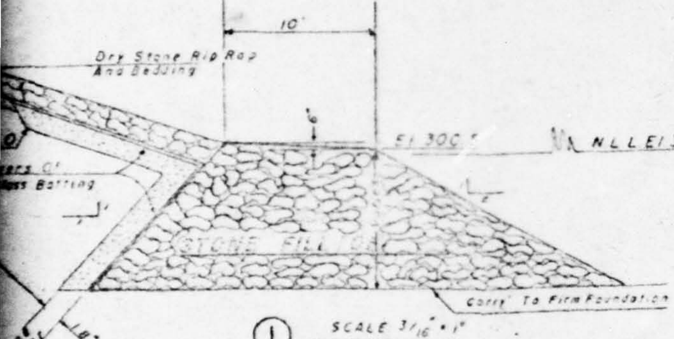


PROFILE- ROCK FILL TOE



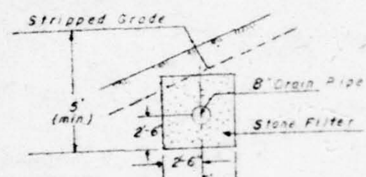
FILTER DRAIN MANHOLE

SCALE 3/8" = 1'-0"



(1) SCALE 3/16" = 1"

PIPE DRAIN - 12" Dia. Porous Wall Concrete Pipe wrapped in Filter - X Cloth



(2) SCALE 3/16" = 1"

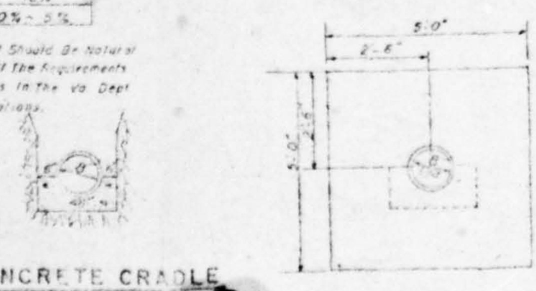
STONE FILTER AND STONE FILL - Meeting The Following Gradations

Sieve Size	*Stone Filter	Stone Fill 4" Max. Size	Stone Fill 6" Max Size (Alternate)
	Percent By Wt Finer Than		
6"			100%
5"			90-100%
4"		100%	70-90%
3"		90-100%	
2"		70 - 80%	50-70%
1"		40-70%	30-50%
0.75"		0 - 40%	0-30%
0.50"		0%	0%
0.25"	100%		
# 4	90% - 100%		
# 8	80% - 100%		
# 16	65% - 85%		
# 30	30% - 60%		
# 50	8% - 26%		
# 100	10%		
# 200	0% - 5%		

*Stone Filter Material Should Be Natural Sands Meeting All Of The Requirements For Fine Aggregates In The Va. Dept Of Hwys Specifications.

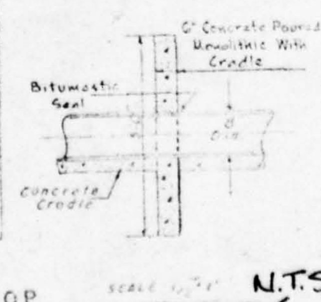
Revision - July 19, 1971 Change in gradation of stone filter material.

Revision - Oct 1, 1971 Change fiberglass waterstop on & ductile iron pipe to concrete
- Delete filter drain pipe and filter material between Sta 5+00 & Sta 6+20
- Show field measurement of rock fill toe in place.



CONCRETE CRADLE

SCALE

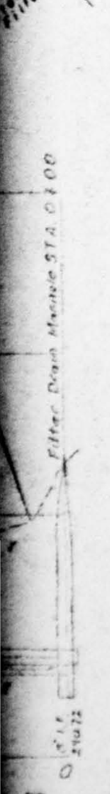


WATER STOP

SCALE

N.T.S.

PLATE 6



UPPER DAM AND LAKE

...UB-SURFACE DRAINS

DATE JULY, 1970 | SCALE AS SHOWN | SHEET 3 OF 3

NOTES

1. Contractor shall verify all sizes, dimensions and location of wall pipes, floor and wall openings, etc. and all bolts for equipment to comply with manufactures recommendations.

2. Reinforcing steel in walls, beams, slabs and footings shall be continuous and the length of splices shall be sufficient to extend well beyond critical section. Lap all bars 40 diameters.

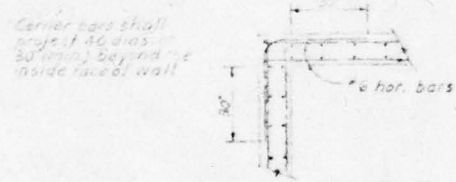
3. Reinforcing steel in slabs, beams and footing shall be supported in the forms by combination spacers and chairs spaced not more than 4'-0" on centers. Reinforcing steel in top of slabs and footings shall be held in place by ½" dia. supporting bars which are secured to suitable chairs spaced not more than 4'-0" on centers.

4. Concrete covering over steel shall be as follows:
   - (a) Deck slab and beams 1½"
   - (b) Outside walls (Lake) 2½"
   - (c) inside walls 2"
   - (d) footings 4"

5. Chamfer all deck beams and exposed edges above Lake Level 1".

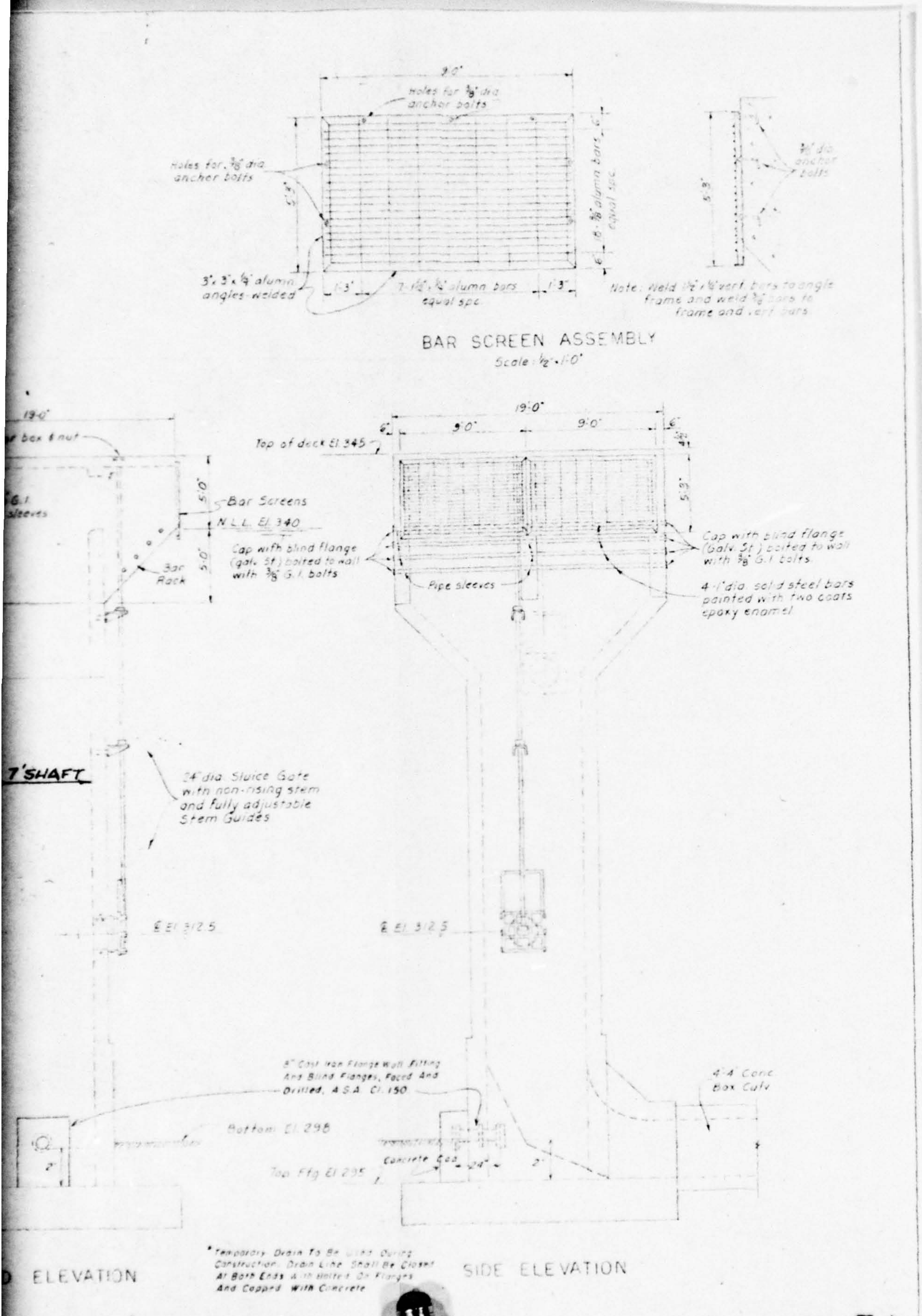
6. Aluminum bars, rods and angles shall conform to A.S.T.M. B 308-67 and B 211-67 or other comparable specifications meeting the engineers approval. The manner and method of welding and the filler material shall meet with the manufactures recommendations. The entire assembly shall have a dull metalic or oxide appearance.

7. All concrete shall be Class A-3 with #57 stone aggregate.



DETAIL OF
CORNER REINFORCEMENT
Scale 3/8" = 1'-0"

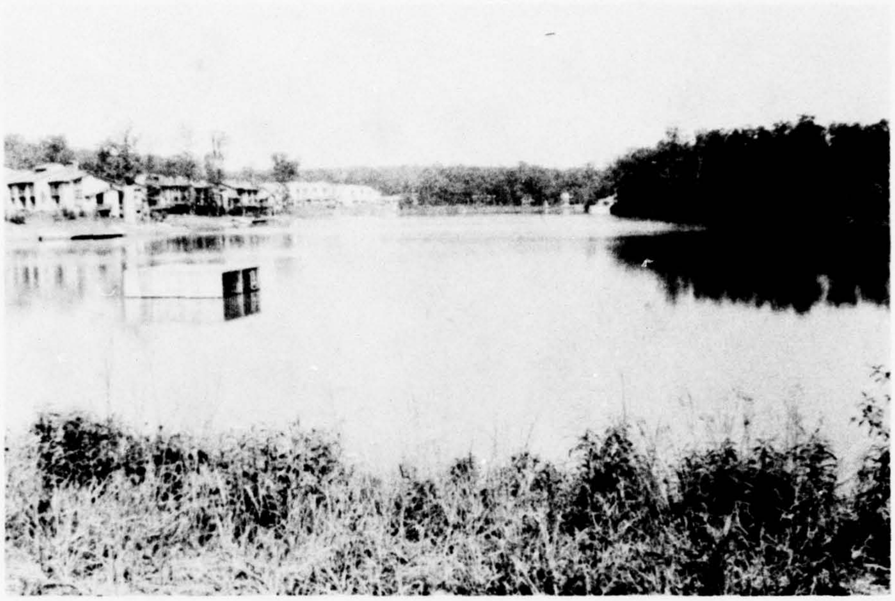
Revision July 19, 1971 Addition Of Temporary Drain


9'-0"
Holes for 3/8" dia anchor bolts
Holes for 3/8" dia anchor bolts
5'-3"
18-3/8" alumn bars equal spc
3" x 3" x 1/4" alumn angles-welded
1'-3"
7-1/2" x 1/4" alumn bars equal spc
1'-3"
5'-3"
3/8" dia anchor bolts
Note: Weld 1/2" x 1/4" vert bars to angle frame and weld 3/8" bars to frame and vert bars
BAR SCREEN ASSEMBLY
Scale: 1/2" = 1'-0"
19'-0"
6"
9'-0"
9'-0"
6"
Top of deck El. 345
5'-3"
Cap with blind flange (Galv. St.) bolted to wall with 3/8" G.I. bolts
Cap with blind flange (galv. St.) bolted to wall with 3/8" G.I. bolts
Pipe sleeves
4-1" dia. solid steel bars painted with two coats epoxy enamel
19'-0"
box & nut
5'-0"
Bar Screens
N.L.L. El. 340
G.I. sleeves
Bar Rack
5'-0"
7' SHAFT
24" dia. Sluice Gate with non-rising stem and fully adjustable Stem Guides
℄ El. 312.5
℄ El. 312.5
8" Cast Iron Flange Wall Fitting And Blind Flanges, Faced And Drilled, A.S.A. Cl. 150
4'-4' Conc Box Culv
Bottom El. 298
Concrete Cap
24"
2'
Top Ftg El. 295
* Temporary Drain To Be Used During Construction. Drain Line Shall Be Closed At Both Ends With Bolted On Flanges And Capped With Concrete
D ELEVATION
SIDE ELEVATION
UPPER DAM & LAKE
PLATE 7
N.T.S.

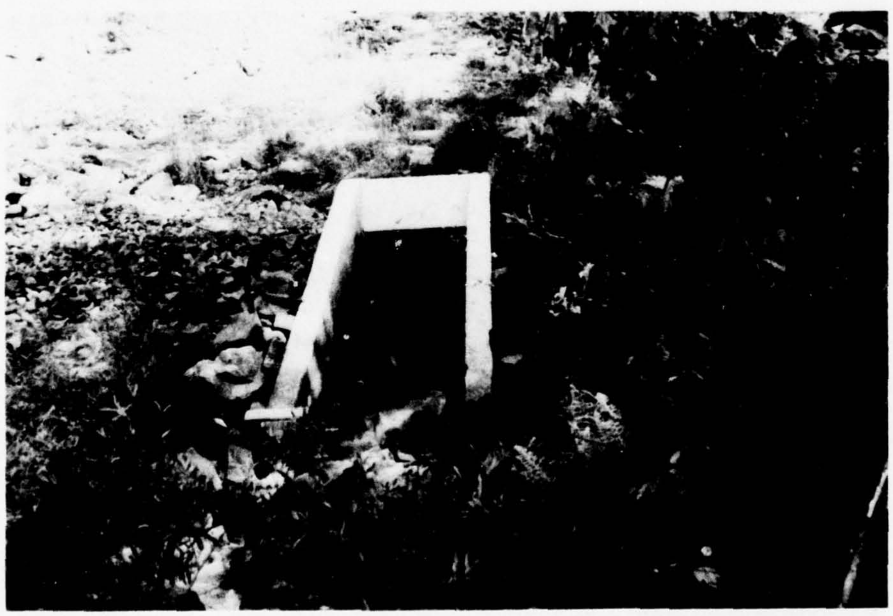
APPENDIX II

PHOTOGRAPHS

UPPER SOUTH LAKE DAM



PHOTOGRAPH NO. 1
Outlet Structure



PHOTOGRAPH NO. 2
Discharge Box Culvert

PHOTOGRAPH NO. 3
Upstream Face of Dam

PHOTOGRAPH NO. 4
Downstream

APPENDIX III

FIELD OBSERVATIONS

Check List
Visual Inspection
Phase I

Name Upper South Lake Dam **County** Fairfax **State** Virginia **Coordinates** 3856.2 Lat.
7719.9 Long.

Date(s) Inspection 5/30/79 **Weather** Sunny **Temperature** 75°F

Pool Elevation at Time of Inspection 340 **M.S.L.** **Tailwater at Time of Inspection** None **M.S.L.**
Will be 300 when Lower Lake is completed.

Inspection Personnel:

Hugh Gildea, SWCB	Charles Chambers, Gulf Reston	
Bert Black, Law Engineering	Riley Chung, Schnabel Engineering Assoc.	
Tan Young, DMM&A		

Paul Seiler, DMM&A **Recorder**

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	No obvious surface cracks.	
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	No obvious cracking at toe.	
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	Near left abutment Seepage is noted below observation well. 1" plastic pipe - water standing 15" above surface 30' horizontal from the top of the dam. Seepage estimated about 5 gallons/minute. Small gullies observed on downstream slope.	Surface erosion along the left abutment due to vehicles driven down the slope. Also there is erosion in the center of the slope.
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST	There are no obvious misalignment of the top of the dam.	
RIPRAP FAILURES	No riprap failures . Macadam path at the top of the riprap which was not installed at the time of inspection.	

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONSTRUCTION MATERIAL	Plans indicate clay type material for cut off of trench.	
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	No obvious cracks at junction of embankment and abutments. Erosion due to car driven down the face of the dam slope.	
ANY NOTICEABLE SEEPAGE	Seepage 30' horizontal from the top of the dam just below the observation well pipes near the left abutment. Seepage is estimated at 3-5 gallons/minute.	
STAFF GAGE AND RECORDER	None.	
DRAINS	There is a horizontal toe drain system which outlets through an 8" diameter pipe to a manhole 290" downstream.	
FOUNDATION	There is a wet area below the dam. This will be covered by the tailwater of the lower lake which is now under construction.	

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	There was no obvious deterioration.	
INTAKE STRUCTURE		
OUTLET STRUCTURE	The outlet structure is a 10 feet diameter riser with a weir at elevation 340. There is a 4' x 4' concrete box from the riser to the toe of the embankment on a 0.503% slope.	
OUTLET CHANNEL	There are rocks on the apron between the wingwalls at the discharge outlet.	
EMERGENCY GATE	N/A	

INSTRUMENTATION

VISUAL EXAMINATION	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	See Appendix I	
OBSERVATION WELLS	None.	
WIERS	None.	
PIEZOMETERS	Five peizometers located about 25 feet toward the downstream from the centerline of the dam.	
OTHER		

RESERVOIR

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Partially forested and partially urbanized. Averaged 4% slope.	
SEDIMENTATION	Unknown.	

DOWNSTREAM CHANNEL

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	Clear.	
SLOPES	Flat.	
APPROXIMATE NO. OF HOMES AND POPULATION	There are over 20 homes on the lake front. Future homes are planned around another lake, i.e. the lower lake which will extend approximately to the toe of this dam. Estimated population immediately adjacent to dam is 60 people.	

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION

ITEM	REMARKS
PLAN OF DAM	See Appendix I - Plans
REGIONAL VICINITY MAP	See Appendix I - Maps.
CONSTRUCTION HISTORY	Built in 1971
TYPICAL SECTIONS OF DAM	See Appendix I - Plans.
HYDROLOGIC/HYDRAULIC DATA	See Section V.
OUTLETS - PLAN and - DETAILS	See Appendix I - Plans.
- CONSTRAINTS and - DISCHARGE RATINGS	
RAINFALL/RESERVOIR RECORDS	None available.

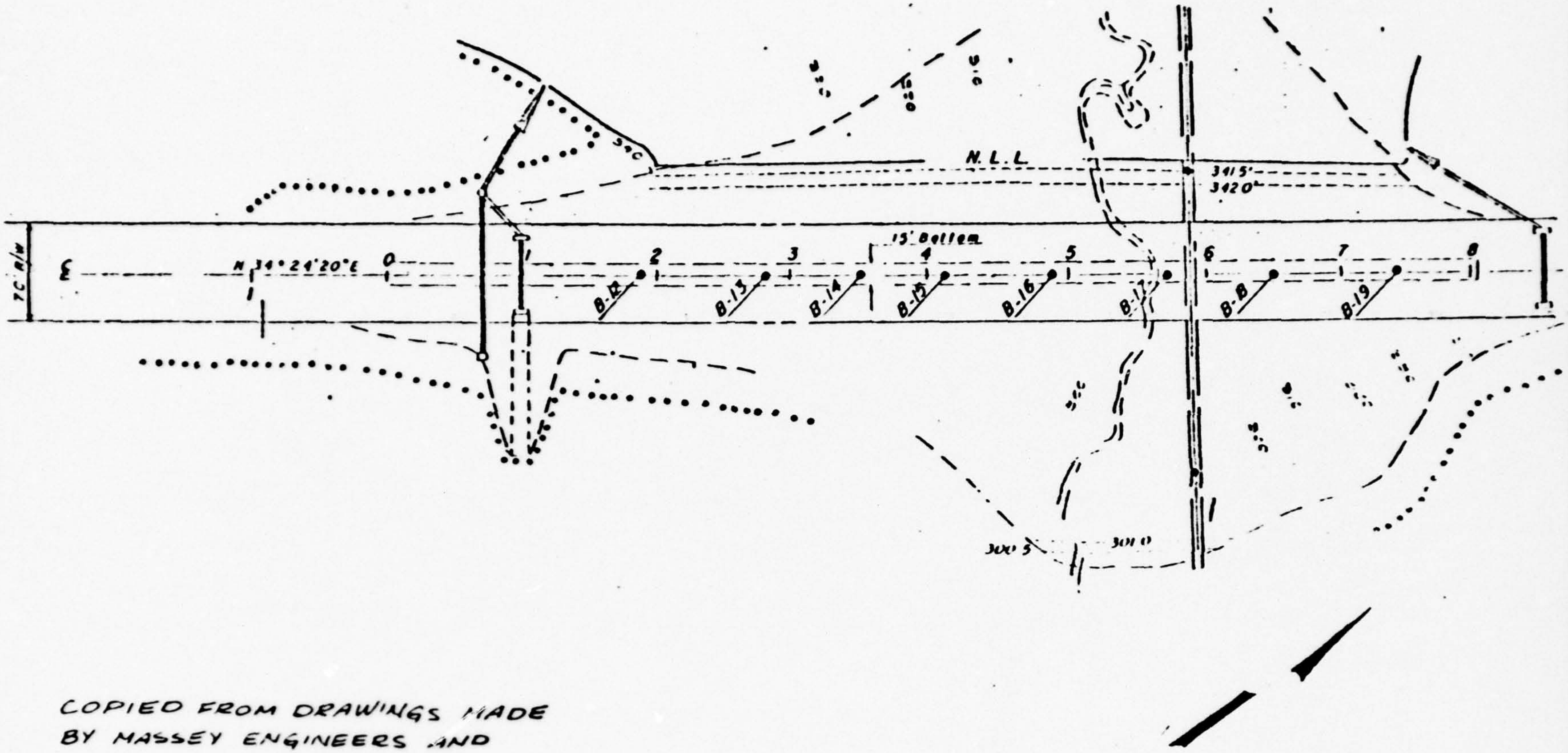
ITEM	REMARKS
DESIGN REPORTS	None available.
GEOLOGY REPORTS	See appendix IV for boring data.
DESIGN COMPUTATIONS	
HYDROLOGY & HYDRAULICS	
DAM STABILITY	None
SEEPAGE STUDIES	1978 by Schnabel Engineering Associates.
MATERIALS INVESTIGATIONS	See Appendices I & IV.
BORING RECORDS	
LABORATORY	
FIELD	
POST-CONSTRUCTION SURVEYS OF DAM	1978 by Schnabel Engineering Associates.
BORROW SOURCES	

ITEM	REMARKS
MONITORING SYSTEMS	None.
MODIFICATIONS	None.
HIGH POOL RECORDS	None available.
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	1978 by Schnabel Engineering Associates
PRIOR ACCIDENTS OF FAILURE OF DAM DESCRIPTION REPORTS	none.
MAINTENANCE OPERATION RECORDS	None.

ITEM	REMARKS
SPILLWAY PLAN	See Appendix I - Plans.
SECTIONS	See Appendix I - Plans.
DETAILS	See Appendix I - Plans.
OPERATING EQUIPMENT PLANS & DETAILS	

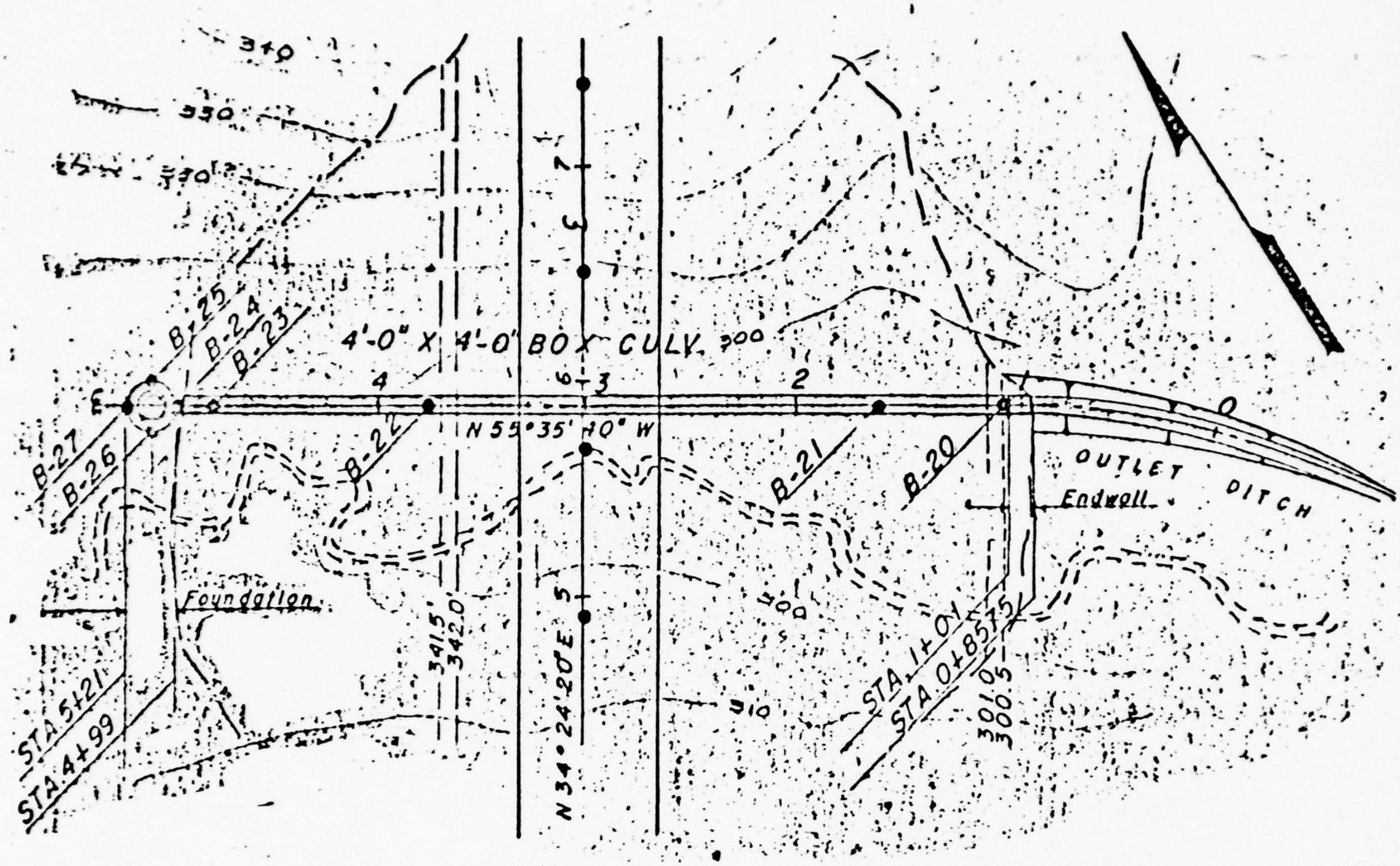
APPENDIX IV

GEOLOGICAL DATA



COPIED FROM DRAWINGS MADE
BY MASSEY ENGINEERS AND
DATED JULY, 1970.

UPPER LAKE DAM	LAW ENGINEERING TESTING COMPANY GEOTECHNICAL & ENVIRONMENTAL CONSULTANTS 7913 WESTPARK DRIVE, MCLEAN, VIRGINIA 22101		
RESTON, VIRGINIA	SCALE 1" = 120' (approx)	Drawn Checked JSJ Date 7-24-79	Job No. MB-2270 Dwg. No. 1



COPIED FROM DRAWINGS
BY MASSEY ENGINEERS
AND DATED JULY, 1970

UPPER LAKE DAM	LAW ENGINEERING TESTING COMPANY GEOTECHNICAL & ENVIRONMENTAL CONSULTANTS 7913 WESTPARK DRIVE, MCLEAN, VIRGINIA 22101		
RESTON, VIRGINIA	SCALE 1" = 75' (approx)	Drawn Checked J.S.J. Date 7-24-79	Job No. WB-2270 Dwg. No. 2

BASED ON LOGS INCLUDED WITH PLANS PREPARED BY MASSEY ENGINEERS AND DATED JULY, 1970.

B-12 GRND ELEV. ±336.99

DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	3	3 4 7	brown clayey silt
2	3		
3	9		
4	12		4'
5	19	78 14 21	brown clayey silt
6	35		
7	42		7'
8	81		brown decomposed rock
8.5	160/6"		
8.8		100/3"	

B-13 GRND ELEV. ±332.97

DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	3	3 5 7	brown clayey silt
2	3		
3	9		
4	12		4'
5	25	10 18 25	brown clayey silt & decomposed rock
6	32		
7	47		
8	69		8'
9	112		brown decomposed rock
9.9		100/3"	

B-14 GRND. ELEV. ±326.12

DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	3	1 3 5	reddish-brown clayey silt
2	3		
3	9		
4	15		4'
5	25	12 20 24	brown clayey sand & quartz
6	35		
7	42		
8	66		8'
9	81		brown decomposed rock
10.5		90 47 61	

B-15 GRND ELEV. ±317.56

DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	2	1 2 4	brown clayey silt
2	4		
3	9		
4	14		4'
5	25	7 7 9	brown clayey silt
6	27		
7	32		
8	40		
9	46		9'
10	51	78 30 41	brown clayey silt & quartz
11	56		
12	80		12'
13	112		brown decomposed rock
13.5	72/6"		
		35 56 70	

UPPER LAKE DAM

RESTON, VIRGINIA

LAW ENGINEERING TESTING COMPANY
GEOTECHNICAL & ENVIRONMENTAL CONSULTANTS
7913 WESTPARK DRIVE, MCLEAN, VIRGINIA 22101

SCALE	Drawn Checked J.S.J. Date 7-24-79	Job No. WE-227
		Dwg. No. 3

B-16 GRND ELEV. ±302.03

DEPTH FT	CASE 0/1'	SPOON 0/6"	FIELD DESCRIPTION
0			0'
1	1	7 / 1 / 4	brown clayey silt
2	2		
3	9		
4	21		4'
5	26	25 / 12 / 10	brown clayey silt & rock fragments
6	35		
7	37		
8	41		
9	60		9'
10	26	35 / 29 / 25	brown silty sand
11	35		
12	52		
13			13'
13.5		70 / 100/3"	brown decomposed rock

B-17 GRND ELEV. ±295.53

DEPTH FT	CASE 0/1'	SPOON 0/6"	FIELD DESCRIPTION
0			0'
1	1	7 / 1 / 1	brown silty clay
2	1		
3	1		
3.5			3.5'
4	9		green decomposed rock
5	20	25 / 33 / 47	
6	25		
7	39		7'
8	39		brown silty clay
9	56		9'
		25 / 56 / 100	brown decomposed rock

B-18 GRND ELEV. ±302.80

DEPTH FT	CASE 0/1'	SPOON 0/6"	FIELD DESCRIPTION
0			0'
1	3	5 / 12 / 18	brown clayey silty decomposed rock
2	12		
3	72		3'
3.9		35 / 100/3"	green decomposed rock

B-19 GRND ELEV. ±336.23

DEPTH FT	CASE 0/1'	SPOON 0/6"	FIELD DESCRIPTION
0			0'
1	3	3 / 4 / 5	brown clayed silt
2	5		
3	18		
4	16		4'
5	49	19 / 16 / 36	yellowish brown clayed silt
5.5	114/6"		5.5'
5.8		100/3"	brown decomposed rock

BASED ON LOGS INCLUDED WITH PLANS PREPARED BY MASSEY ENGINEERS AND DATED JULY, 1970.

UPPER LAKE DAM	LAW ENGINEERING TESTING COMPANY GEOTECHNICAL & ENVIRONMENTAL CONSULTANTS 7913 WESTPARK DRIVE, MCLEAN, VIRGINIA 22101		
RESTON, VIRGINIA	SCALE	Drawn Checked J.S.J. Date 7-24-79	Job No. WB-2270 Dwg No. 4

BASED ON LOGS
INCLUDED W/ PLANS
PREPARED BY MASSEY
ENGINEERS AND
DATED JULY, 1970.

B-20 GRND ELEV. ±295.57

DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	5	3 7 12	brown silty clay & decomposed rock
2	12		
3	33		
4	61		4'
4.9		36 100/3"	brown decomposed rock

B-21 GRND ELEV. ±299.65

DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	3	3 4 6	gray micaceous clayey silt
2	4		
3	6		
4	9		4'
5	25	25 30 42	brown silty clay & quartz rock
6	42		
7	65		
8	78		
9	95		9'
		27 42 86	green decomposed rock

B-22 GRND ELEV. ±293.79

DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	2	1 2 3	topsoil
2	3		
3	14		
4	19		4'
5	21	7 9 10	brown silty clay & quartz
6	29		
7	19		
8	29		
9	37		9'
10	52	10 12 12	brown decomposed rock
11	75		
12	81		
13	120		13'
13.8		66 100/4"	brown decomposed rock

B-23 GRND ELEV. ±299.59

DEPTH FT	CASE b/1'	SPOON b/6"	FIELD DESCRIPTION
0			0'
1	3	3 3 5	brown silty sandy clay
2	4		
3	6		
4	9		4'
5	11	11 10 10	brown clayey silt & decomposed rock
6	11		
7	10		
8	25		8'
9	51		brown decomposed rock
		37 51 30	

UPPER LAKE DAM	LAW ENGINEERING TESTING COMPANY GEOTECHNICAL & ENVIRONMENTAL CONSULTANTS 7913 WESTPARK DRIVE, MCLEAN, VIRGINIA 22101		
RESTON, VIRGINIA	SCALE	Drawn Checked J.S.J Date 7-24-79	Job No WB 227 Dwg No 5

BASED ON LOGS INCLUDED WITH PLANS PREPARED BY MASSEY ENGINEERS AND DATED JULY, 1970.

B-24 GRND ELEV. ±300.05

DEPTH FT	CASE 0/1'	SPOON 0/6"	FIELD DESCRIPTION
0			0'
1	2	2 2 9	brown sandy clay
2	3		
3	6		
4	9		4'
5	18	12 11 14	brown decomposed rock
6	14		
7	15		
8	16		
9	81		9'
10	46	35 37 43	brown sandy clay & decomposed rock
11	57		
12	92		
12.3	80/3"	100/3"	12.3'
			gray decomposed rock

B-25 GRND ELEV. ±301.08

DEPTH FT	CASE 0/1'	SPOON 0/6"	FIELD DESCRIPTION
0			0'
1	2	2 6 4	brown silty clay
2	7		
3	6		
4	7		4'
5	9	6 7 16	brown sandy silt & decomposed rock
6	13		
7	26		
8	35		8'
9	109		brown decomposed rock
9.8		76 100/3"	

B-26 GRND ELEV. ±299.39

DEPTH FT	CASE 0/1'	SPOON 0/6"	FIELD DESCRIPTION
0			0'
1	1	1 1	brown silty clay
2	2		
3	2		
4	15		4'
5	17	18 16 16	brown decomposed rock & quartz
6	27		
7	18		
8	15		
9	40		9'
10	27	25 69 68	brown decomposed rock
11	93		11'
12	120		brown decomposed rock
12.5	27/6"		
		92 100/3"	

B-27 GRND ELEV. ±299.90

DEPTH FT	CASE 0/1'	SPOON 0/6"	FIELD DESCRIPTION
0			0'
1	3	8 3 5	brown silty clay
2	2		
3	3		
4	11		4'
5	14	13 14 22	brown sandy silt & decomposed rock
6	18		
7	10		
8	10		8'
9	47		brown decomposed rock
9.8		100/3"	

UPPER LAKE DAM

RESTON, VIRGINIA

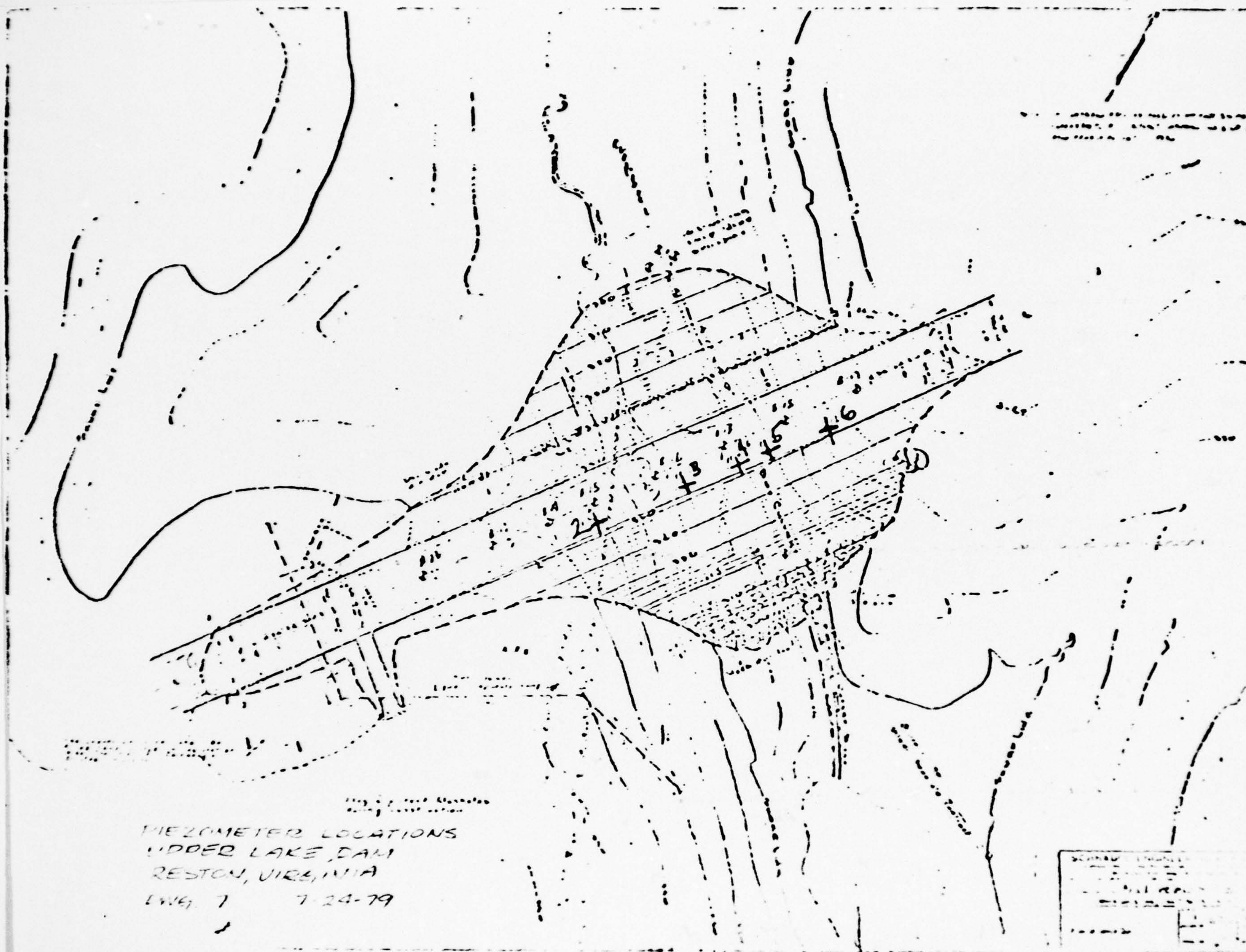
LAW ENGINEERING TESTING COMPANY
GEOTECHNICAL & ENVIRONMENTAL CONSULTANTS
7913 WESTPARK DRIVE, MCLEAN, VIRGINIA 22101

SCALE

Drawn
Checked JSJ
Date 7-24-79

Job No. B-2270

Dwg No. 6



PIEZOMETER LOCATIONS
UPPER LAKE DAM
RESTON, VIRGINIA
DWG. 7 7-24-79

Water Loss or Gain During Drilling

Loss or gain of water during drilling at each test boring location was noted in the test boring logs. The loss of water occured almost always at the contact surface between various subsurface strata, particularly at the contact between Stratum C and Strata B and D (disintegrated rock and highly weathered and highly fractured phyllite rock, respectively). Loss of water was also indicated just below the contact of Stratum A (embankment fill) and Stratum B (natural soils) in Boring PZ-13. Both PZ-7 and PZ-8 indicated the loss of water while drilling through Strata B and C. However, water was regained in both holes when coring in Stratum C (rock).

The amount of flow seeping out of the area located just east of PZ-11 was monitored closely during the drilling of PZ-11. The flow was steady during drilling to a depth of 27 feet. At the same time, no water was observed within the hole. However, the seepage flow was immediately increased as soon as the drilling proceeded below 27 feet. From the test boring log, the 27-foot depth is about 2 feet below the top of the disintegrated rock surface. Apparently below this level joints are present in the rock which permitted the drilling water to flow laterally to a critical point where an upward flow erosion channel had been established in the past under the high peizometric pressure from the reservoir. The drilling water was then forced up under this additional pressure head and exited together with seepage water at the leakage point.

Field Permeability Test Results

Variable head field permeability tests of the rock and the disintegrated rock of Strata D and C were conducted at the completion of each test boring prior to piezometer installation. In most cases, casings were driven to near the top of the Stratum C material (disintegrated rock). Water in each boring was bailed to a depth as low as possible. In Borings PZ-7, 8 and 11, the inflow of water after bailing was so fast that it made the bailing extremely difficult. However, a reasonable differential head was introduced to permit testing.

The test results were evaluated by using various formulae listed on Page 7-4-9 of MAVFAC DM 7 Manual. These tests were conducted with 3-1/2 inch casings seated into the upper regions of disintegrated rock of Stratum C, and after each hole was completed to the lowest level penetrated. The computed coefficients of permeability, which represent average permeability values of both Strata C and D rock, were as follows:

Boring No.	Coefficient of Pemreability, k (cm/sec)
PZ-7	6.0×10^{-3}
PZ-8	5.1×10^{-3}
PZ-9	3.1×10^{-4}
PZ-11	2.4×10^{-2}
PZ-12	3.4×10^{-2}
PZ-13	9.9×10^{-3}
Average all tests	8.1×10^{-3}

Data on Materials

Available data on the embankment material properties has been summarized in the May 25, 1978 report from Schnabel to Gulf Reston. This summary is as follows:

Graduation Test

Unified Soil Classification:	ML-MH predominantly
Percent Passing No. 200 Sieve:	69 to 80%
Liquid Limit:	40 to 53%
Plastic Index:	9 to 21%

(One sample indicated to be nonplastic)

Density Test

Laboratory (ASTM D-698, Method A)

Average Max. Dry Density	= 105 pcf
Optimum Moisture Content	= 18%

Field

Average Dry Density	= 100 pcf
Average Moisture Content	= 20%
Average Relative Compaction	= 95%

APPENDIX V

SUMMARY AND CONCLUSION OF PHASE I & PHASE III
SCHNABEL ENGINEERING ASSOCIATES

I. SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS - PHASE I

Based on the engineering data contained in this report, the following summary of conclusions and recommendations is presented.

a. The upper dam of Lake Elsa may be considered in the intermediate category since its maximum height is 60 ft. We considered the dam to be of a high hazard classification as there would be excessive and extensive economic loss to the community both upstream and downstream of the dam.

b. The contract drawings and technical specifications for dam were reviewed for conformance with generally accepted principles of engineering practice. Our review of design and construction documents, and visual inspection of the dam did not reveal any serious deficiencies, except the following:

1. Minor transverse cracking over downstream embankment surface.
2. Suspected minor underseepage problem near and below downstream toe of the dam.
3. The sand filter bedding is located at the surface instead of being below the riprap at the downstream slope.
4. Rocks accumulated within the end wall structure of box culvert.
5. Minor erosion at the upper elevation of upstream slope protection.
6. A spring located at about El 330 at the interface of the embankment and left abutment which is suspected to be the seepage water from reservoir.

c. The lake level rising study made by Massey Engineers-Consultants indicates that the drop inlet spillway has the capacity of passing either a 100-year flood or a problem maximum flood (PMF) without overtopping the dam. However, under a PMF condition the reservoir will rise up to a level to cause first floor flooding for houses along the shoreline. The dam will also only have 0.88 ft left as freeboard under this PMF condition.

d. Recommended measures for correcting the unfavorable conditions are presented in Section VI of the report. These recommendations are summarized as follows:

1. Transverse cracking should be sealed off by itself once the settlement is completed. These crackings will not impose any danger for this low head, wide crest, and relatively flat sloped homogeneous dam.

2. Install 2 to 3 piezometers on the downstream toe to observe the exit water pressure in order that the existence of excessive pressures in the rock joints can be either verified or disproved.

3. Resurface and redress the downstream slope including the areas where transverse crackings occurred and the riprap slope protection and filter bedding.

4. Raise the upstream riprap by one foot to El 342.5 for slope protection against wave action.

e. The recommended procedures to locate and verify the source of the spring at El 330 are presented in section VII of this report. The corrective measures including field exploration and grouting operation are beyond the scope of this study.

f. The following recommendations and further studies were also suggested in relation to the lake level raising study.

1. Raise the crest elevation of the dam to El 351.77 such that a 1.88 ft freeboard may be attained for the dam under a PMF condition.

2. Study the possibility of constructing additional discharging facilities, e.g. an emergency spillway, to prevent the houses from flooding under a PMF condition.

g. A risk analysis should be considered on the lower dam and reservoir area considering the most severe condition, i.e., if the upper dam were overtopped.

We have prepared this report in accordance with generally accepted geotechnical engineering practices, and make no other warranties, either expressed or implied, as to the professional services provided under the terms of this agreement and included in this report.

I. SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS - PHASE II

We have reached the following summary of conclusions and recommendations based upon the engineering data collected during Phase II investigation and presented in this report.

a. The source of leakage at left abutment has been determined from the drilling of PZ-11, which is located within the embankment and is about 3.5 feet higher in elevation than that of the "spring". The interconnection of seepage water below the disintegrated rock between PZ-11 and the "spring" caused the drilling water and dye water in PZ-11 to flow out of the "spring" under excessive water pressure from the reservoir. The lake is therefore confirmed to be the source of this leakage.

b. The existence of seepage underneath the dam through the highly fractured foundation rock has been verified from the following investigation results:

1. Almost equal piezometric levels were recorded in piezometers located at the crest just upstream and downstream from the core trench. This indicates the ineffectiveness of the installed shallow core trench.

2. The excessive water pressure at PZ-11 which was about 1.5 feet above the existing ground surface.

3. Piezometric levels recorded in PZ-7 and 8 indicated about 1 to 2 feet above the base of the rockfill toe.

4. The general wetness and softness at areas downstream from the rockfill toe.

5. The numerous locations in the downstream valley where the dye water was observed. The dye water was injected into the foundation rock at PZ-9 located upstream at the crest.

c. We have also evaluated the engineering properties of both soil and rock materials by conducting field and laboratory testing. Their results

are presented in Sections III and V of this report. Both embankment fill and overburden soils may be classified as sandy clayey silt to sandy silt or type ML according to ASTM D-2487. The materials also have an average permeability value of 1.5×10^{-5} cm/sec. The underlying foundation rock is phyllite rock of Wissahickon formation. The rock is generally highly weathered and highly fractured to the depths investigated. Furthermore, the rock also possess a prominent foliation which strikes northwest or about perpendicular to the dam axis. Field permeability tests indicated an average coefficient of permeability of 8.1×10^{-3} cm/sec for this highly fractured rock mass, or at least 100 times more permeable than the embankment.

d. Remedial measures for this upper dam are presented in Section VII. The remedial work should include the installation of a grout curtain in the left abutment and central portions of the dam at the location shown on Sheet 3, and the area grouting around the leakage point if required. General requirements of grout holes, grout materials, and grouting procedures were included in the same section.

e. Other corrective measures, as addressed in our Phase I report, to correct the unfavorable physical appearance of the dam should also be implemented.

We have prepared this report in accordance with generally accepted geotechnical engineering practices, and make no other warranties, either expressed or implied, as to the professional services provided under the terms of this agreement and included in this report.

We would recommend that the project specifications for remedial work contain the following statement:

"A geotechnical engineering report has been prepared for this project by Schnabel Engineering Associates using the guideline of Dam Safety Inspection Program prepared by Corps of Engineers. This report is available to prospective bidders for informational purposes only and should not be considered as part of the contract documents. The opinions expressed in this report are those of the geotechnical engineer and represent his interpretation of the subsurface conditions, field observations, tests, and the results of analyses which he has conducted. Should the data contained in this report not be adequate for the contractor's purposes, the contractor may make his own investigations, tests, and analyses for use in his bid preparations. The report may be examined by bidders at the owner's office or copies may be procured from Schnabel Engineering Associates at nominal charge."

Test boring data as shown by Drawings V78391-1 and -2 should be included in the contract documents and made available to prospective bidders.

APPENDIX VI

REFERENCES

LIST OF REFERENCES

1. Recommended Guidelines for Safety Inspection of Dams, Department of the Army, Office of the Chief of Engineers, Washington, D.C. 20314

2. HEC-I, Flood Hydrograph Package, Hydrologic Engineering Center, U S Army, Corps of Engineers, Davis, California 1973.

3. U S Weather Bureau and U S Army Corps of Engineers, Seasonal Variation of Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 1,000 Square Miles and Durations of 6, 12, 24 and 48 Hours, Hydrmeteorological Report No. 33, Washington, D.C., April 1956.